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Stress history study of the Allegheny River bridge (Pennsylvania turnpike) /

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STRESS HISTORY STUDY OF THE ALLEGHENY RIVER BRIDGE

(PENNSYLVANIA TURNPIKE)

by

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ABSTRACT

Strain measurements were taken at several structural steel details of the Allegheny River Bridge under normal traffic conditions. Previous inspection of the bridge by Pennsylvania Turnpike personnel had revealed fatigue cracks in the tie plates connecting the floor beams to the outrigger cantilever brackets. Because of this the emphasis of the study was on these plates. Strain gages were mounted on four tie plates on a floor beam and on the longitudinal girders. An automatic data acquisition system was used to record the strain range occurrences and an analog trace recorder was used to determine live load strain variations with time. Stress ranges in the girders were comparable to measured values obtained by other investigators from other girder bridges. Horizontal in-plane bending stresses were observed in the tie plates, with magnitudes of these stresses two to three times as high as those in the girders. These horizontal bending stresses were caused by the elongating and shortening of the top flange of girders due to truck traffic.

A model was developed to describe the tie plate behavior. Two boundary conditions were assumed for the model: a simply supported case and a fixed-fixed case. The measured stresses at the tie plates fall in-between values computed from the model with these two boundary conditions. The spectrum of measured strains in several tie plates

and an estimate of truck traffic during the life of the bridge were used to estimate the cumulative damage in these tie plates by Miner's Hypothesis. A comparison between the root-mean-square (RMS) stress range and constant cycle laboratory fatigue test data on riveted joints was also made. Results of analysis by both the RMS procedure and the Miner's Hypothesis explained the existence of the fatigue cracks at the tie plates. Further analysis by three dimensional models are needed for a more accurate description of the tie plate behavior under traffic load. Also a program to evaluate the fatigue crack propagation at rivet holes due to bending is needed.

1. INTRODUCTION

1.1 Background and Objectives

Inspection of steel highway bridges in the United States has resulted in the detection of fatigue cracks at certain structural details. Details such as the ends of cover plates, web and flange attachments and tie plates connecting transverse floor beams and brackets have exhibited fatigue cracks at weld toe terminations, tack welds or bolt holes. The Yellow Mill Pond Bridge on the Connecticut Turnpike(1), the Lehigh River and Canal Bridges on U. S. Route 22 in Pennsylvania(2), and the Allegheny River Bridge on the Pennsylvania Turnpike are among bridges where some of these cracks have been found. All of these bridges are located in urban areas and carry very high volumes of truck traffic.

Test data obtained to define the fatigue strength of coverplated beams has indicated that the "threshold" level for fatigue is near 5 ksi(12). The crack growth threshold is not well defined for a large class of detail, particularly riveted connections.

The field testing of the Allegheny River Bridge provided an opportunity to gather data for a stress-history study of some bridge details under in-service conditions. The bridge is located on a heavily traveled artery, the Pennsylvania Turnpike, outside the city of Pittsburgh. The bridge also contained tie plate details which had experienced detectable fatigue crack growth.

This stress history study presents the method of data acquisition, the stresses observed, vehicular travel on the bridge, the fatigue cracks and their possible causes, correlation between laboratory and field test results, and predictions of the life of the replaced tie plates.

1.2 Description of Bridge

The Allegheny River Bridge, Fig. 1, is composed of a 4-span continuous beam-girder bridge and a 5-span truss bridge. The bridge carries both east and westbound traffic. The tie plate details, some of which developed fatigue cracks, were located in the 4-span continuous beam-girder bridge. An end span and part of a second span (see Fig. 2) were chosen for testing because of their ac essibility.

The plan and elevation of the 104'-4" end span and the 130'-5" second span are shown in Fig. 3. The longitudinal girders are 7 ft. 1/2 in. deep in most of the test spans except where they are haunched at the piers to 9 ft. 1-1/2 in. A typical cross-section of the bridge is shown in Fig. 4a. The end span has a load carrying steel system composed of 11 floor beams (web: 66 in. x 3/8 in., flanges: 2 angles 6" x 6" x 1/2" with outrigger brackets and 12 stringers (W12 X 62). The end span contains a hinge, located 78 ft. 3 in. from the west abutment (see Fig. 3). The second span has a similar load carrying system. A typical (the original) tie plate detail (14" x 1/2" x 4' 7-15/16") is shown in Fig. 4b.

Preliminary inspections in the fall of 1971 by Pennsylvania Turnpike Commission personnel revealed several cracks in the tie plates. Upon further inspection in January of 1972, all tie plates were again checked for fatigue cracks. The approximate locations and length of the cracks that were in the test spans is given in Fig. 5. Some rivets connecting the tie plates to the first outboard or first inboard stringer were found to be ineffective as a result of either stretching of the shank, fracture through the shank or popped rivet heads. These rivets are indicated by the darkened circles in Fig. 5.

Appendix A shows that most of the cracks were at or near the piers and abutments. All cracks originated from rivet holes; most cracks forming where the tie plates were connected to the main longitudinal girders.

In the spring of 1972, reinforcement tie plates (17" x 1/2" x 4' 7-15/16") were added to the structure. The original cracked tie plates were groove welded at the crack locations and the reinforcement plates were placed on top of them. The bolt holes in the reinforcement plates were matched to those of the original plates for ease of installment. At two locations, the original tie plate was removed for examination and only the reinforcement plate alone is in place. Inservice testing was conducted in the fall of 1972.

2. STRAIN DATA ACQUISITION

2.1 Strain Gages

Electrical resistance strain gages were mounted on the reinforcement tie plates, on the longitudinal girders, and on a floor beam to obtain stresses under traffic. Eighteen gages on the reinforcement tie plates were located where several fatigue cracks had been previously discovered in the original tie plate. The approximate location of the gages are schematically shown in Fig. 6. These gages, placed parallel to the edges of the tie plates, were generally located on the outboard stringer side of the tie plate connection to the main girder as illustrated in Fig. 7.

Three strain gages were placed on the main girders: one on the top flange of the right (eastbound) girder near the haunches, the second on the bottom near the abutment at the end of the cover plate of the same girder. The third gage was placed on the bottom of the left girder, near the abutment at the end of the cover plate and symmetrical to the gage on the right girder (see Fig. 8). Two strain gages, located directly below stiffeners, were placed on the bottom of a floor beam.

The gages used were 1/4 in. long electrical resistance gages of the foil type. Moisture and other environmental effects to the gages were prevented by application of weather-proof coatings. To minimize the effect of temperature changes, the gages were connected to temperature compensating gages and plates.

2.2 Recording Systems

Two independent systems were used to monitor strains due to traffic: the FHWA automatic data acquisition system and an ultraviolet analog trace recorder.

The FHWA system was used to monitor strain ranges at the gage locations over extended time intervals. Located in a van, the system shown in Fig. 9, consists of an amplifier, an analog-to-digital converter, a computer and a teletype machine(3). Prior to monitoring, ten strain range levels were chosen for each of the ten gage locations being monitored simultaneously. A test level (the tenth level) was also defined for each gage to exclude very low strains due to vibration and automobile and light-weight truck traffic. A time period (for example, one hour) was selected to print out the data. The computer then began monitoring. As a vehicle traveled across the bridge, the variation of strain at each of the ten gage locations were amplified and the magnitudes of the strain ranges were stored in the computer. The number of strain range occurrences between two chosen strain levels during the period were then printed out as in Fig. 10. To reduce the effect of drifting of the zero level of strains, the level was checked periodically during the monitoring periods. This helped insure accurate recording of the strain ranges.

An analog trace recorder was also used to gather data on typical strain variations due to traffic. Figure 11 shows a typical analog trace of live-load strain magnitudes as a function of time. The analog recorder and the FHWA system could monitor several gages simultaneously. Correlation of data was checked by simultaneous recordings of the systems for short periods (16 minutes) of time.

2.3 Traffic Identification

Visual observation and recording of traffic flows were undertaken for short periods of time in conjunction with analog trace recording. Traffic on the bridge could then be correlated with recorded strains in the tie plates, girders and floor beam.

The truck classification and their sketches are shown in Fig. 12. Some traffic vehicles (that is, passenger automobiles and panel trucks) were excluded because they generated very small strains.

2.4 Strain Recording Period

This study was undertaken from November 10, 1972 to November 17, 1972. The chronological record of strain data acquisition is shown in Fig. 13. A total of 143 hours of strain data were acquired for the stress history.

3. STRESS MAGNITUDES

3.1 Stresses in the Reinforcement and Original Tie Plates

The cracks in the original tie plates were repaired by groove welds. The rivets were removed and reinforcement plates placed on top of the original plates. Both plates were then bolted to the flange of the floor beams and the outrigger brackets as well as to the first inboard and outboard stringers and the main girders, as shown in Fig. 14. Stresses obtained at the gaged tie plate locations reflected this composite plate behavior provided by the original and the reinforcement plates, except at floor beam No. 0, left where the original plate was removed for examination.

The analog traces of strain variations indicated that each truck crossing the bridge caused a stress range occurrence at all gage locations. The recorded strains in most of the tie plates were much higher than those observed in the girders and the floor beam.

Assuming a modulus of elasticity for steel of thirty million pounds per square inch, recorded strain values were converted linearly tp stresses. The maximum live load stresses in the tie plates, as recorded by the analog traces, are shown in Table 1. The highest recorded stress was 18.6 ksi. Most of the recorded stresses were caused by a single truck, but occasionally two trucks were seen traveling together or in different directions but came to the gage locations at about the same time.

Table 1 also shows the highest (first) strain range levels selected for the FHWA system. Stress range is defined as the difference between a maximum stress and the following minimum stress. Gage locations subjected to high maximum live load stresses also showed high stress ranges.

Figures 15 to 19 show the recorded stress distribution in individual tie plates at a given time. The approximate locations of the gages on the tie plates are also shown in the figures. Figure 16 gives the stress distribution on the reinforcement tie plate at L - 0 (westbound) on the inboard stringer side of the girder. Figure 17 gives the stress distribution on the same tie plate on the outboard stringer side of the girder. The stress distributions in this tie plate indicate that the plate was subjected to high bending stresses in the horizontal plane. The directions (or signs) of the stresses to both sides of the longitudinal girder suggested that the horizontal bending was induced by a relative movement between the girder and the ends of the tie plate. Stress distributions were about the same at other tie plates. Only small values of stress were recorded at the centerline of the tie plates, indicating small axial elongation or vertical bending.

Superimposition on the plan-view sketch of the tie plates of the stress distribution caused by a truck at each of the gaged tie plates is shown in Fig. 20. From the stresses obtained for these plates the maximum horizontal bending stresses were higher near the abutment and pier. This agrees with the crack pattern observed in the tie plates (see Fig. 5). Figure 20 also indicates that stresses in the tie plates were dependent on the location of the tie plates in the bridge.

3.2 Stresses in the Girders and in a Floor Beam

Very low live load stresses were observed in the main girders and in a fllor beam of the Allegheny River Bridge. A summary of the maximum live load stresses at the gaged locations on the girder flanges and the floor beam, as recorded by the analog system, is given in Table 2. The table also shows the highest stress range levels selected for the FHWA system for the gages on the girders and floor beam. The maximum live load stress magnitudes in the girders were in the order of 2 to 3 ksi with maximum stress ranges between 3 and 5 ksi. These recorded values are comparable to results reported by other investigators on main longitudinal members of shorter spans[3,4,5].

The main girder design stresses were found by loading an HS20-44 truck in one lane of the bridge. Gage 21, located near floor beam 1 on the bottom of the left main girder, experienced maximum live load stresses of about 2.4 ksi and stress ranges in the order of 4.65 ksi during the field studies. Both these magnitudes of stress and stress range fall below the calculated design live load stress of 9.6 ksi.

The time variation of stresses on a point of a girder due to a truck is shown in Fig. 21. The superposition of the static response and vibrational stresses gives the total stresses. A similarity exists between the static stress variation for a location on a girder flange and the stress influence line for that location. The analog recording for gage 21 in Fig. 21 is analogous to the stress influence line for a point in the left span of the 4-span continuous beam of Fig. 22. This indicates that the stresses in the girders were generated by the truck traffic. The measured live load stresses in the floor beam were practically zero.

4. STRESS EVALUATION

Figure 23 shows the strain variation traces at gage 12, located on a tie plate, and gage 20, located at a point on the girder, due to various types of trucks traveling across the bridge. (For truck types, see Fig. 12). The traces show that each truck crossing the bridge caused a stress range excursion at the tie plates and the girders. The traces also indicate that the stress-time pattern at a tie plate on the bridge was the same for all types of trucks, only the magnitude of the strain range changed. This suggests a direct relationship between the strain variation in the tie plates and the strain occurring in the longitudinal girders.

4.1 Model of Analysis

In the stress evaluation of the Lehigh Canal Bridge[2] a preliminary model was assumed to describe the behavior of the tie plates due to a vehicle crossing the entire bridge. Horizontal bending in the tie plates would be caused by any longitudinal displacement at the top flange of the girder. A two-dimensional model was developed to describe more accurately the tie plate behavior in the Allegheny River Bridge using the concept of displacement induced moment. This model is also valid for the Lehigh Canal Bridge and other bridges having similar geometry and tie plate configurations.

The model, shown in Fig. 24, assumes the floor beam to be attached to the inboard and outboard stringers and the girder. Longitudinal displacement, δ, of the girder will cause horizontal bending to occur in the tie plates, which are located over the girder. To estimate the magnitude of horizontal bending stresses in the tie plates, the boundary conditions at the first inboard and outboard stringers are assumed to be simply supported for one case, shown in Fig. 25a and fixed at both ends for the second case, Fig. 25b. The bending moment in the tie plate at the edge of the girder is $M_B = \alpha\,\delta$, where α is determined by the geometry of the tie plate and floor beam and the boundary conditions. The actual bending moment lies between the bounds of the assumed cases of simply supported and fixed ended.

The longitudinal displacement, δ, of a point on the top flange of the girder is the product of the slope, θ, of the deflection curve of the girder and the distance from the neutral axis to the top flange, c. $\delta = c\,\theta$. The slope of the girder due to traffic over the bridge can be estimated from the influence line for slope at the point of interest. The change of slope as a truck travels across the bridge causes back and forth rotation, θ_r, and longitudinal displacement, δ_r, of the top flange causing horizontal bending in the tie plate. The range of horizontal bending moment is therefore, $M_{br} = \alpha\,c\,\theta_r$.

4.2 Results of Analysis

The approximate influence line for the slope of the girder at the abutment of the bridge was computed and is shown in Fig. 26. The change in slope is the sum of the maximum and minimum slope, $\delta_r = \frac{(82,200 + 40,000)}{E\ I}$. By using the values of E I and the centroidal distance to the plate, c, and assuming that the girder sustains the total weight of an HS20-44 truck, the range of horizontal bending moment for the two boundary conditions of Fig. 25 were found to be 404 kip-in. and 1141 kip-in. for the simply supported and the fixed-ended cases. The corresponding stress ranges at a gage location are 13.4 ksi and 30.3 ksi for the two cases. The maximum measured stress range at this location was 18.6 ksi. Thus, the conditions of the model give a good estimate of the measured stresses. When the results of a more accurate three-dimensional model are available the agreement between the measured and the computed stresses is expected to be even better.

Since the strain in the tie plate is linearly proportional to the horizontal bending moment ($\varepsilon = \alpha\ c\ \theta$), the strain in the tie plate is linearly proportional to the girder slope. Thus the strain influence line for the tie plates is analogous to the influence lines for slope in the longitudinal girder at the tie plate location. Figure 27 shows a comparison between the strain variation for gage 15, located on tie plate 3A-R under the eastbound lane, with the approximate slope of the deflection curve at the same location on the girder. It is apparent that these two curves are compatible with each other.

For a more complete and accurate evaluation of the Allegheny River Bridge, a three-dimensional analysis should be undertaken.

5. STRESS RANGE OCCURRENCES

An example of data output from the FHWA system is shown in Fig. 10. The system was programmed to record the number of strain range occurrences between predetermined magnitudes of strain ranges. These magnitudes of strain ranges are converted to stress range levels assuming E = 30,000 ksi. Table 3 lists the stress range levels and the number of stress range occurrences between these levels. Observations showed that one vehicle causes one stress range occurrence. Gage 5, for example, was subjected only once to a stress range greater than 11.7 ksi. It had ten occurrences between 10.5 and 11.7 ksi; thirty occurrences between 9.3 and 10.5 ksi; etc. The total number of stress range occurrences for gage 5 were 4.429 over a recording period of 34 hours and 14 minutes. Sometimes the highest (first) strain levels were not set high enough, although these are a very small percentage of the total number of occurrences. Gages located on the same tie plate and having similar recording times, experienced different totals of stress range occurrences. For example, gages 5 and 8, located on the same tie plate and having the same recording time (34 hours and 14 minutes) experienced 4,429 and 28,984 stress range occurrences respectively. This is because gage 5 had a larger number of stress range occurrences below the threshold of 0.9 ksi than did gage 8. The same is true for the other gages on the same tie plates.

A summary of the stress range data for the gages on the girders and the floor beam is given in Table 4.

The stress range occurrence data in Tables 3 and 4 were plotted as histograms. The percentage of frequency of occurrence between the stress range levels for gages 3, 11, 15, 18 (on the tie plates) and 20 and 21 (on the girders) are shown as examples of the histograms in Figs. 28 to 33. The lowest one or two stress range intervals were not plotted in all but one of the histograms (gage 21). Some observations could be made although the recording periods were not very long. A large percentage of occurrences for the tie plates were observed at very low stress ranges (below 4.5 ksi). All tie plates recorded high stress ranges in the order of 12 to 18 ksi.

For gages 3 and 11, located on tie plates at the abutment, a larger percentage of higher stress ranges were observed than for gage 15, located near the hinge in the bridge. Gage 18, located on a tie plate near the pier also experienced more occurrences at higher stress range levels. The gages on the main longitudinal members (gages 20 and 21) had relatively low stress range levels. The double-peaked histograms for the girders indicates the effects of 2-axle and empty trucks and larger and heavy trucks, respectively. Only the larger and heavier trucks induced stresses above 1.8 to 2.4 ksi.

6. TRAFFIC RECORDS

6.1 Traffic Counts

Traffic counts were taken on the Allegheny River Bridge during the in-service testing period, on four consecutive week days for 40, 60, 100 and 19 minutes respectively. The results are summarized in Table 5. The highest volume of trucks consisted of 5-axle tractor semi-trailers (3S-2). The percentage of the different types of trucks traveling in the eastbound direction was almost the same as those traveling in the westbound direction.

The Pennsylvania Turnpike Commission classifies vehicle traffic by weight. Table 6 lists the truck traffic flow over the bridge during the field testing period (11/10/72 - 11/17/72). Table 7 lists the frequency of occurrence of various truck types during the same period. The high percentage of vehicles between 19,000 and 80,000 lbs. compares with the high percentage of five and four axle tractor semi-trailers observed in Table 5. The results from Table 7 are plotted as gross vehicle weight vs. frequency histograms for the eastbound and westbound directions in Figs. 34 and 35. Both of these histograms show two peaks, indicating large numbers of loaded trucks at 62 to 80 kips and large numbers of loaded smaller trucks (2D, 3) and unloaded tractor semi-trailers at 19 to 30 kips. These records are also comparable to results from loadometer surveys throughout the state and nation.

6.2 Loadometer Surveys

Figure 36 gives the results of the 1972 PennDOT loadometer survey for twenty stations located on main arteries throughout the state. The gross vehicle weight histogram has two peaks, one at 60 to 75 kips and the other at 24 to 36 kips. This is comparable to the results obtained for the Allegheny River Bridge during the in-service testing period.

The results of the 1970 FHWA Nationwide Loadometer survey are given in Fig. 37. There are two peaks in the gross vehicle weight histogram at approximately 25 and 70 kips. This compares with the results from the Allegheny River Bridge as well as the twenty station survey from Pennsylvania.

The general agreement of the Pennsylvania Turnpike records for the Allegheny River Bridge with the twenty station PennDOT and nationwide records indicates that the Turnpike records can be considered representative of the actual nominal truck traffic over similar structures. The vehicle induced stresses in the tie plates can also be considered nominal. The stress frequencies will be correlated with traffic records in the next chapter for the evaluation of the fatigue cracks.

7. CORRELATION OF TRAFFIC AND STRESS DATA WITH FATIGUE TEST RESULTS

7.1 Fatigue Test Data

Recent fatigue tests on beams and girders established that constant amplitude fatigue test data can be represented by straight lines on a log-log plot of stress range vs. number of cycles to failure. The slopes of these lines for different types of details on beams and girders is practically the same.(6) Hansen(7) and Baron and Larson(8) performed laboratory fatigue tests on riveted joints under constant stress range cycles in tension. The specimens from these two test series had similar rivet configurations as the tie plates on the Allegheny River Bridge. Figure 38 shows the data from the two test series plotted as a function of stress range and cycle life. The solid line in the figure is plotted using the mean values of the test data and the slope of the line representing the fatigue characteristics of cover-plated beams.

In order to evaluate the fatigue cracks in the original tie plates through correlating the measured stress spectra of the tie plates with constant amplitude fatigue test results from the riveted joints, adjustments must be made to estimate the stresses at the rivet holes of the originally cracked plates. First, adjustments of stresses are made to the original tie plate from the measured values in the reinforcement plates. The factor of adjustment for each tie plate was derived from the boundary conditions of the model shown in Figs. 25a and 25b. The stress was then adjusted to the edge of the rivet hole. A third adjustment must be made to account for the stress concentration at these rivet holes caused by horizontal bending. A factor of 2.5 was assumed based on the stress concentration at rivet holes under tensile loading which is approximately 2.5 to 3.8 times the nominal stress[9].

To compare the random stress range variations at the rivet holes with constant stress-range cycle fatigue data, two methods can be used. They are the root-mean-square (RMS) method and Miner's Hypothesis.

7.2 Root-Mean-Square Estimates

The root-mean-square (RMS) method[10,11,12] weighs the stress ranges in a spectrum and converts the spectrum into an equivalent constant amplitude cyclic stress range which is correlated with the number of cycles corresponding to the spectrum.

The stress range spectra of the gages on the tie plates are given in Table 3. The spectrum at any gage was adjusted, as described above, to the edge of the rivet hole. The RMS values of the adjusted stress ranges above an estimated crack growth threshold level of 4 to 5 ksi[13] were evaluated for a number of gages and are presented in Table 8 as $S_{r_{RMS}}$ at the rivet hole. The cumulative frequencies of stress range occurrences for the stress ranges that were above the crack growth threshold are also given in the table.

The total number of commercial vehicles that have traveled over the bridge during the twenty-one years it has been in service (1952 - 1972) was provided by the Pennsylvania Turnpike Commission. The total volume of truck traffic up to and including 1972 for trucks above a weight of 19 kips was found to be 20.7 million vehicles. This corresponds to a constant growth rate of 2.3% since 1952. Assuming that each truck caused one stress excursion, each tie plate would have been subjected to 20.7 million cycles of loading. Neglecting stresses below the crack growth threshold, the number of cycles (the RMS stress range cycles) causing damage is less than 20.7 million. The estimated number of RMS stress range cycles for several tie plates are given in the last column of Table 8.

The S_{rRMS} at the rivet hole and the corresponding fatigue cycles for the tie plates summarized in Table 8 are plotted in Fig. 38 and compared with the mean (solid) line based on the laboratory results of Hansen, and Baron and Larson. All the data points lie on or to the right of this line. Since all the tie plates experienced fatigue cracks at the rivets, this is as expected. Figure 38 indicates that the RMS stress range provides good correlation between the field test data and the laboratory test results. Laboratory fatigue tests in bending should be undertaken on tie plates with the same geometry and rivet configuration found on the Allegheny River Bridge to give a more accurate simulation of the tie plate behavior.

7.3 Correlation by Miner's Hypothesis

The adjusted stress range spectra at the rivet holes of the tie plates with gages 3, 11, 15 and 18 were used to estimate the cumulative fatigue damage in each plate by Miner's Cumulative Damage Hypothesis[14].

The number of stress range cycles at a stress range above the threshold value, n, is computed by the percentage frequency of occurrence at the stress range multiplied by the total truck traffic of 20.7 million during the twenty-one years of bridge service. Values of the number of cycles, N, which would cause failure at each stress range interval were calculated by using the equation for the mean line in Fig. 38 ($\log N_R = 9.98827 - 3.0 \log S_r$). The sum of the ratio n/N for all stress ranges above the threshold are listed in Table 9 for four tie plates. By Miner's Hypothesis, fatigue cracks would develop if the value of the sum is higher than 1.0. For all four tie plates in the table, the value is much higher than 1.0, thus fatigue cracks would be expected, and all the tie plates did experience fatigue cracks.

The results of Table 9 indicate that Miner's Hypothesis can also be used to correlate field measurements and laboratory test results in terms of stress range and cycle life.

7.4 Fatigue Life Prediction of Reinforcement and Original Plates

The cracked original tie plates were repaired by groove welding. The rivets were then burned off and the reinforcement plates placed on top of the original ones. The original and reinforcement plates were then bolted to the girders, floor beams, outrigger brackets and the first inboard and outboard stringers. The lock nuts of the carbon steel bolts were considered not effective so all locks were tack welded.

From the model of analysis in section 4.2, increasing the width of the tie plates will cause the moment induced by the elongating and shortening of the girder to be more severe as shown in Fig. 39. The stresses at the edge of the reinforcement tie plate will also be higher than those in the original plates as shown in Fig. 40. In addition, the depositing of tack welds at the lock nuts created a relatively weak structural detail with respect to fatigue.

Since the cause of longitudinal displacement at the tie plates has not been eliminated, horizontal bending stresses at the edge of the reinforcement plates and at the tack welds could be expected to be as high as or higher than those on the original tie plates. Consequently, under extremely high volumes of truck traffic across the bridge, fatigue cracks may be anticipated to initiate from the tack weld as it did in the Lehigh Canal Bridge[2]. Periodic inspection of the tie plates is therefore recommended.

8. SUMMARY AND CONCLUSIONS

The following summary and conclusions can be drawn from the field studies and the analyses of the girder-and-floor beam spans of the Allegheny River Bridge:

1. The stresses of the main girders under normal traffic were lower than the design live load stresses for the members. The magnitude of the observed stresses were similar to those observed in longitudinal beams and girders of other bridges. The live load stress variations due to traffic could be evaluated through common procedures of structural analysis.

2. Normal stresses of relatively high magnitude (18 ksi) were measured at the tie plates connecting the outrigger brackets and floor beams over the main girders. The stresses in the tie plates were much higher than those observed in the main longitudinal members.

3. Stress distribution in the tie plates indicated longitudinal bending of the plates. The pattern of stress distribution was not affected by the type of trucks but the magnitudes of the stresses were affected. The stress distribution pattern suggested

that the horizontal bending of the tie plate was induced by the longitudinal displacement of the top flange of the main girder under load.

4. A model was developed to evaluate the bending stresses in the tie plate. The longitudinal displacement at the top flange of the girder was estimated from the influence line for slope of the girder at the tie plates. The stresses in the tie plate were then calculated considering the out-rigger bracket and the floor beam as a continuous beam. Calculated stresses agreed fairly well with measured values.

5. The pattern of the measured stresses agreed with the crack pattern and location in the tie plates.

6. The measured stress history at the bridge was compatible with the frequency distribution of trucks reported by the Pennsylvania Turnpike Commission. The stress history also was compatible to the distribution reported by PennDOT on comparable roads, as well as to the records of a nationwide study.

7. Measured stresses in the repair reinforcement tie plates were converted to stresses in the original tie plates for the evaluation of fatigue cracks.

8. The root-mean-square (RMS) procedure was used in correlating stresses in the tie plates and the fatigue crack. Root-mean-square stress range (S_{rRMS}) above the estimated fatigue crack threshold, and the corresponding truck traffic volume from records of the Pennsylvania Turnpike Commission were compared with a S-N curve obtained in the laboratory for similar riveted joints. The RMS procedure showed good correlation with the laboratory test results.

9. Miner's Hypothesis was also used and provided an estimate of the damage that occurred in the tie plates.

10. Periodic inspections should be made to check the possibility of fatigue cracks occurring at the tack welds placed during the repair of the original plates and addition of the reinforcement plates.

11. A comprehensive laboratory testing program is needed to collect data on the effects of tie plates due to horizontal in-plane bending. Also, it is necessary that tests be undertaken to evaluate the stress concentration at rivet holes due to bending stresses.

TABLE 1 - STRESS IN THE TIE PLATES

Gage	Maximum Live Load Stress(1) (by Analog Traces) (ksi)	Highest Stress Range Level(2) (FHWA System) (ksi)
1	10.8	11.7
2	1.2	--
3	7.2	11.7
4	8.4	14.4
5*	11.4	11.7
6*	6.0	--
7*	14.1	14.4
8*	18.6	22.5
9*	13.8	17.1
10*	2.4	--
11*	14.1	17.1
12*	16.8	17.1
13	7.5	14.4
14	0.9	--
15	6.6	11.7
16	11.4	17.1
17	0.6	--
18	10.8	14.4

(1) Maximum live load stresses at gage locations due to a truck crossing the bridge.

(2) Highest stress range level selected for counting of stress range occurrences.

*Original plate removed, gages only on replacement plate.

TABLE 2 - STRESSES IN GIRDERS AND FLOOR BEAM

Gage Location	Maximum Live Load Stress[(1)] (by Analog Traces) (ksi)	Highest Stress Range Level[(2)] (FHWA System) (ksi)
19 Girder R Top	0.9	1.95
20 Girder R Bot.	2.7	3.30
21 Girder L Bot.	2.4	4.65
23 Floor Beam 1	1.8	3.60

(1) Maximum live load stresses at gage locations due to a truck crossing the bridge.

(2) Highest stress range level selected for counting of stress range occurrences.

TABLE 3 STRESS RANGE OCCURRENCES

TIE PLATES

Level	Gage 1		Gage 3		Gage 4		Gage 5		Gage 7	
	S_r (ksi)	N_i	S_r (ksi)	N_i	S_r (ksi)	N_i	S_r (ksi)	N_i	S_r (ksi)	N_i
		5		1		0		1		64
1	11.7		11.7		14.4		11.7		14.4	
		6		2		4		10		147
2	10.5		10.5		12.9		10.5		12.9	
		17		13		15		30		312
3	9.3		9.3		11.4		9.3		11.4	
		15		75		101		38		413
4	8.1		8.1		9.9		8.1		9.9	
		16		232		294		11		387
5	6.9		6.9		8.4		6.9		8.4	
		21		339		442		17		352
6	5.7		5.7		6.9		5.7		6.9	
		98		344		383		15		360
7	4.5		4.5		5.4		4.5		5.4	
		677		501		472		47		552
8	3.3		3.3		3.9		3.3		3.9	
		4350		1624		1802		1163		2213
9	2.1		2.1		2.4		2.1		2.4	
		6385		1985		4104		3097		8140
Min.	0.9				0.9		0.9		0.9	
Total N		11,590		5116		7617		4429		12,940
Recorded Hr: Min	35:43		34:43		35:43		34:14		61:27	

TABLE 3 STRESS RANGE OCCURRENCES

TIE PLATES (continued)

Level	Gage 8		Gage 9		Gage 11		Gage 12		Gage 13	
	S_r (ksi)	N_i	S_r (ksi)	N_i	S_r (ksi)	N_i	S_r (ksi)	N_i	S_r (ksi)	N_i
		0		5		11		12		5
1	22.5		17.1		17.1		17.1		14.4	
		1		12		78		71		6
2	20.1		15.3		15.3		15.3		12.9	
		71		43		234		245		51
3	17.7		13.5		13.5		13.5		11.4	
		334		82		387		407		160
4	15.3		11.7		11.7		11.7		9.9	
		354		66		423		421		248
5	12.9		9.9		9.9		9.9		8.4	
		332		61		417		392		434
6	10.5		8.1		8.1		8.1		6.9	
		309		41		432		423		694
7	8.1		6.3		6.3		6.3		5.4	
		632		107		766		494		1101
8	5.7		4.5		4.5		4.5		3.9	
		4697		2745		3261		3770		3430
9	3.3		2.7		2.7		2.7		2.4	
		22,254		14,407		16,482		17, 305		5708
Min	0.9		0.9		0.9		0.9		0.9	
Total N		28,984		17,569		22,491		23,840		11,837
Recorded Hr: Min	34:14		43:59		43:59		43:59		45:28	

TABLE 3 STRESS RANGE OCCURRENCES

TIE PLATES (continued)

Level	Gage 15		Gage 16		Gage 18	
	S_r (ksi)	N_i	S_r (ksi)	N_i	S_r (ksi)	N_i
		4		0		14
1	11.7		17.7		14.4	
		7		7		25
2	10.5		15.3		12.9	
		38		50		105
3	9.3		13.5		11.4	
		95		173		261
4	8.1		11.7		9.9	
		250		199		358
5	6.9		9.9		8.4	
		409		164		560
6	5.7		8.1		6.9	
		799		164		932
7	4.5		6.3		5.4	
		1429		428		1766
8	3.3		4.5		3.9	
		3474		1693		5085
9	2.1		2.7		2.4	
		5323		5010		9161
Min	0.9		0.9		0.9	
Total N		11,828		7888		18,267
Recorded Hr: Min	20:09		20:09		47:22	

TABLE 4

STRESS RANGE OCCURRENCES: GIRDERS AND FLOOR BEAM

Level	Gage 19		Gage 20		Gage 21		Gage 22	
	S_r (ksi)	N_i	S_r (ksi)	N_i	S_r (ksi)	N_i	S_r (ksi)	N_i
		0		3		0		0
1	1.95		3.3		4.65		3.6	
		1		12		3		0
2	1.80		3.0		4.2		3.3	
		4		44		16		1
3	1.65		2.7		3.75		3.0	
		6		135		41		2
4	1.50		2.4		3.3		2.7	
		13		252		75		4
5	1.35		2.1		2.85		2.4	
		75		288		72		5
6	1.20		1.8		2.4		2.1	
		94		243		46		14
7	1.05		1.5		1.95		1.8	
		116		303		53		26
8	0.90		1.2		1.5		1.5	
		135		481		54		30
9	0.75		0.9		1.05		1.2	
		131		398		44		23
Min	0.60		0.6		0.9		0.9	
Total N		575		2159		404		105
Recorded Hr: Min	35:19		25:19		7:57		16:08	

TABLE 5 - ALLEGHENY RIVER BRIDGE TRAFFIC COUNT DATA

FIELD STUDY DATA

	MONDAY 11/13/72 40 Min.	TUESDAY 11/14/72 60 Min.		WEDNESDAY 11/15/72 100 Min.		THURSDAY 11/16/72 19 Min.		TOTAL TRAFFIC		TOTAL TRAFFIC (%)	
	East	East	West	East	West	East	West	East	West	East	West
2D	15	16	6	26	29	4	5	61	40	16.31	12.90
3	1	0	3	8	4	3	0	12	7	3.21	2.26
2S-1	1	3	5	10	11	1	2	15	18	4.01	5.81
2S-2	7	18	18	24	25	2	5	51	48	13.64	15.48
3S-2	33	51	53	126	129	25	15	235	197	62.83	63.55
Total	57	88	85	194	198	35	27	374	310	100.00	100.00

TABLE 6

TRAFFIC FLOW OVER THE ALLEGHENY RIVER BRIDGE (11/10/72 - 11/17/72)

(PENNSYLVANIA TURNPIKE COMMISSION) (24 HRS.)

WEIGHT (kips)	FRIDAY 11/10/72		SATURDAY 11/11/72		SUNDAY 11/12/72		MONDAY 11/13/72		TUESDAY 11/14/72		WEDNESDAY 11/15/72		THURSDAY 11/16/72		FRIDAY 11/17/72	
	East	West	East	West	East	West	East	West	East	West	East	West	East	West	East	West
19-30	460	594	173	212	58	81	388	525	438	518	454	487	433	561	461	532
30-45	323	331	158	184	104	108	281	275	344	290	359	317	400	373	335	340
45-62	383	320	252	170	168	131	359	262	355	329	459	341	407	331	401	322
62-80	508	484	297	241	191	237	385	506	487	609	562	619	537	626	534	522
80-100	21	18	17	7	6	8	8	13	16	18	22	18	18	18	15	12
Over 100	0	0	0	0	0	0	0	4	0	2	1	0	0	0	0	0
Total	1695	1747	897	814	527	565	1421	1585	1640	1764	1672	1782	1795	1909	1747	1728

TABLE 7 - FREQUENCY OF TRAFFIC FLOW OVER

THE ALLEGHENY RIVER BRIDGE

(11/10/72 - 11/17/72)

WEIGHT (kips)	Total East	Total West	Total East (%)	Total West (%)
19-30	2865	3508	24.74	29.49
30-45	2304	2218	19.90	18.65
45-62	2784	2206	24.04	18.55
62-80	3502	3844	30.25	32.32
80-100	123	112	1.06	0.94
over 100	1	6	0.01	0.05
TOTAL	11,579	11,894	100.00	100.00

TABLE 8

ROOT-MEAN-SQUARE STRESS RANGE CORRELATION OF STRESS AND CYCLE DATA

TIE PLATES

Plate	Gage	Adjusted S_{rRMS} at Rivet Hole (ksi)	Cumulative Frequency (% above Threshold)	n Stress Cycles $\times 10^6$
ABUT RT	3	9.9	61.20	12.7
ABUT LT	11	14.4	26.72	5.5
3A RT	15	9.2	55.00	11.4
5 RT	18	11.3	49.86	10.3

TABLE 9

CORRELATION BY MINER'S HYPOTHESIS

Plate	Gage	$\Sigma \frac{n^*}{N}$ (Test Data)	Remarks
ABUT RT	3	1.64	Cracked at Rivet
ABUT LT	11	2.47	Cracked at Rivet
3A RT	15	1.16	Cracked at Rivet
5 RT	18	2.10	Cracked at Rivet

*N determined from estimate of mean.

Fig. 1 Allegheny River Bridge (Top View)

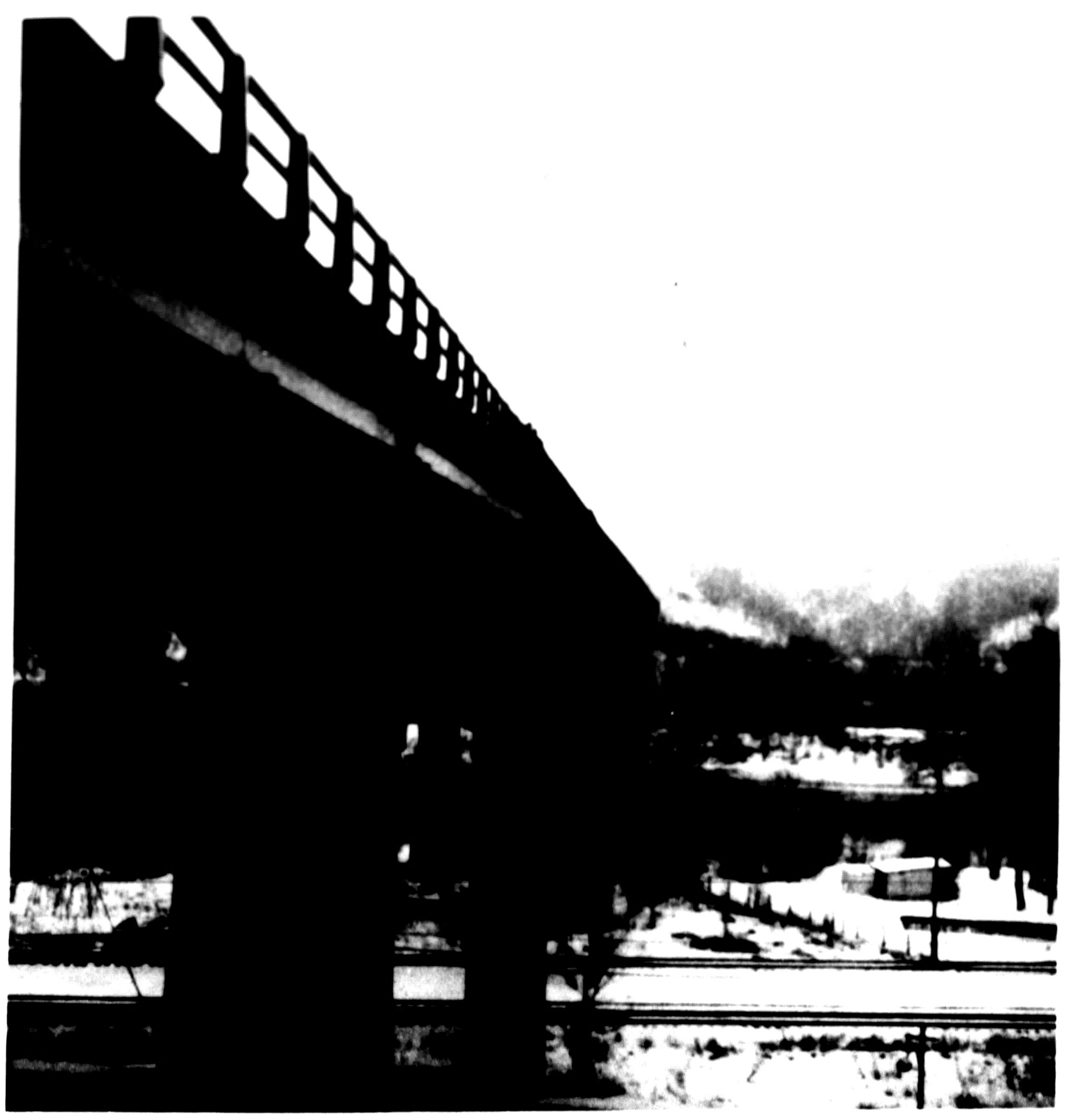


Fig. 2 Allegheny River Bridge (Side View)

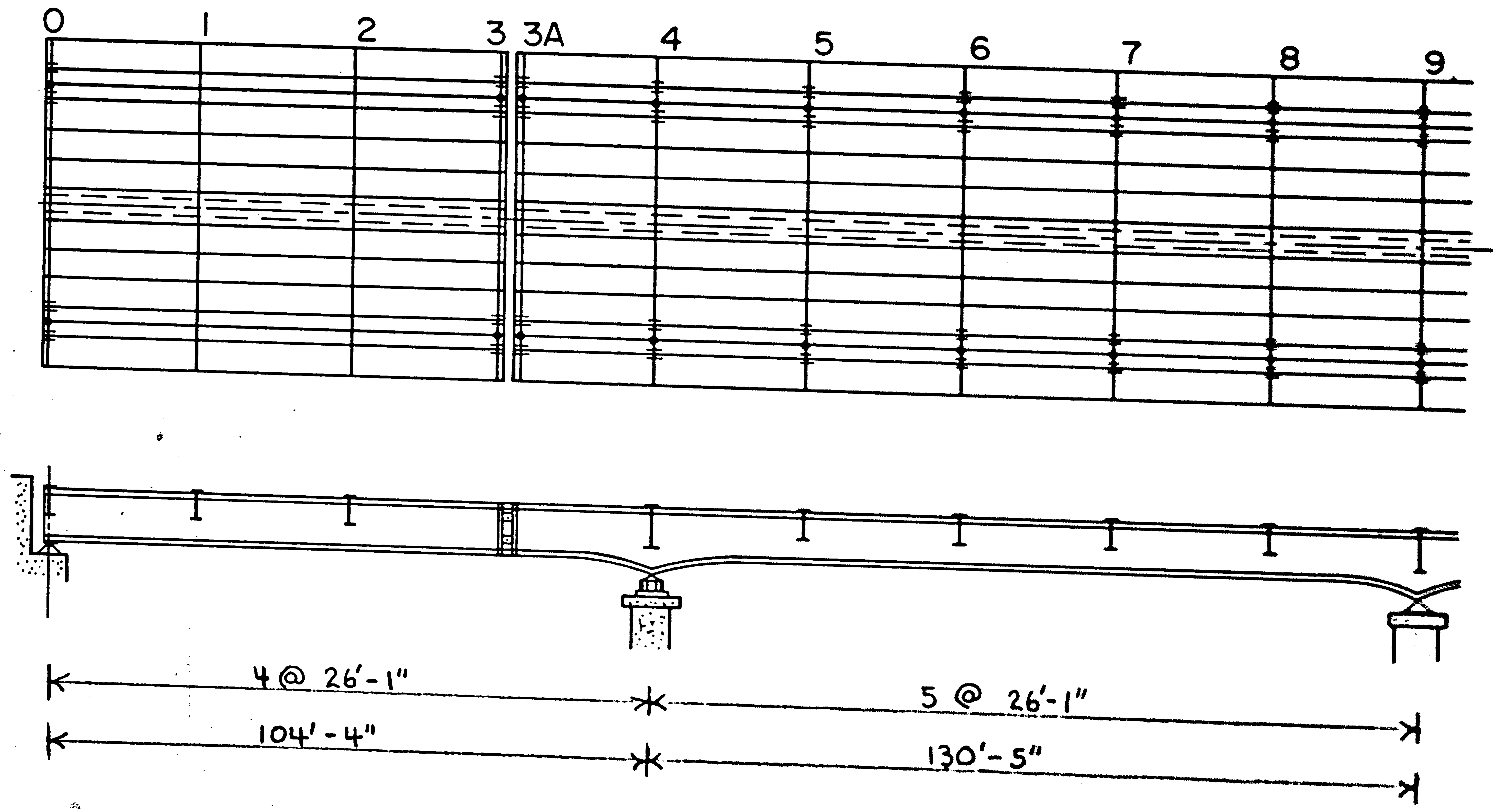


Fig. 3 Plan and Elevation of Test Spans

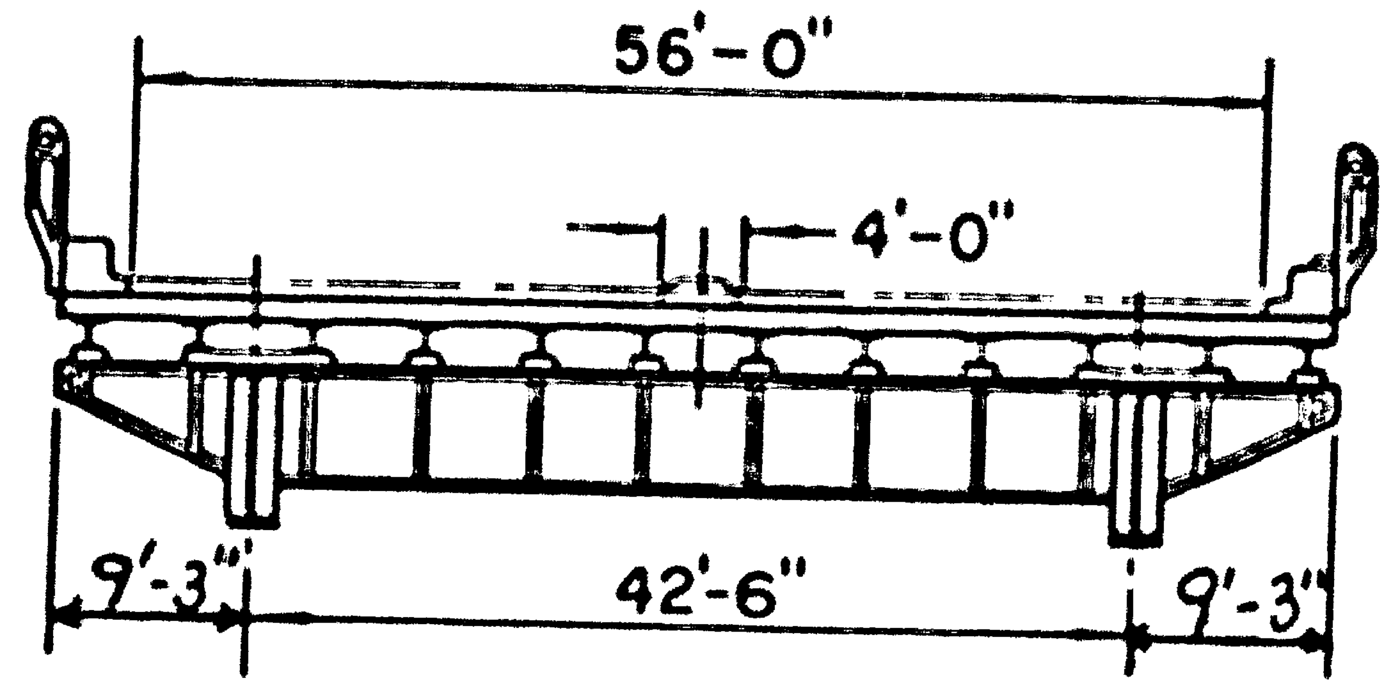


Fig. 4a Bridge Cross Section

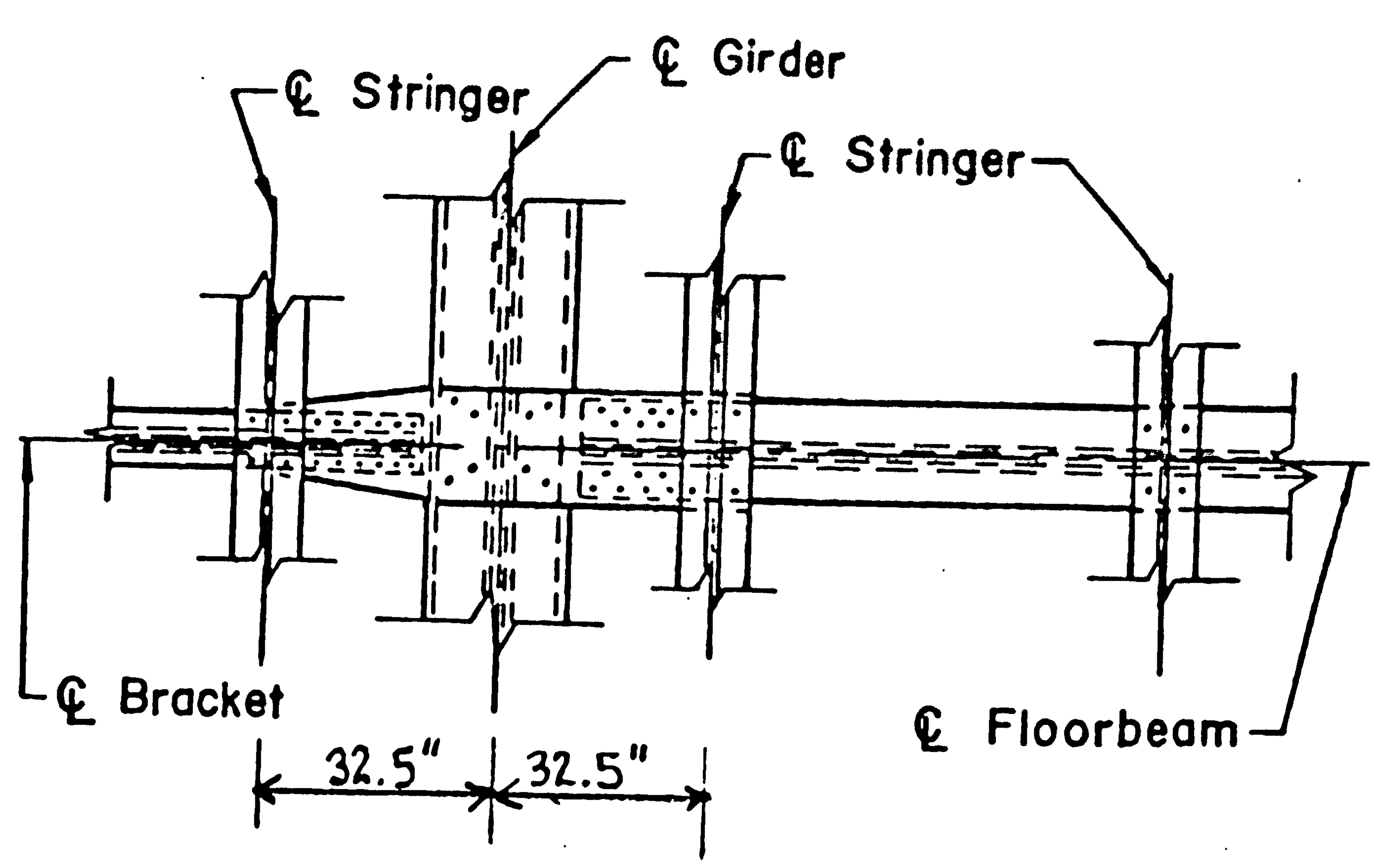


Fig. 4b Original Tie-Plate Detail

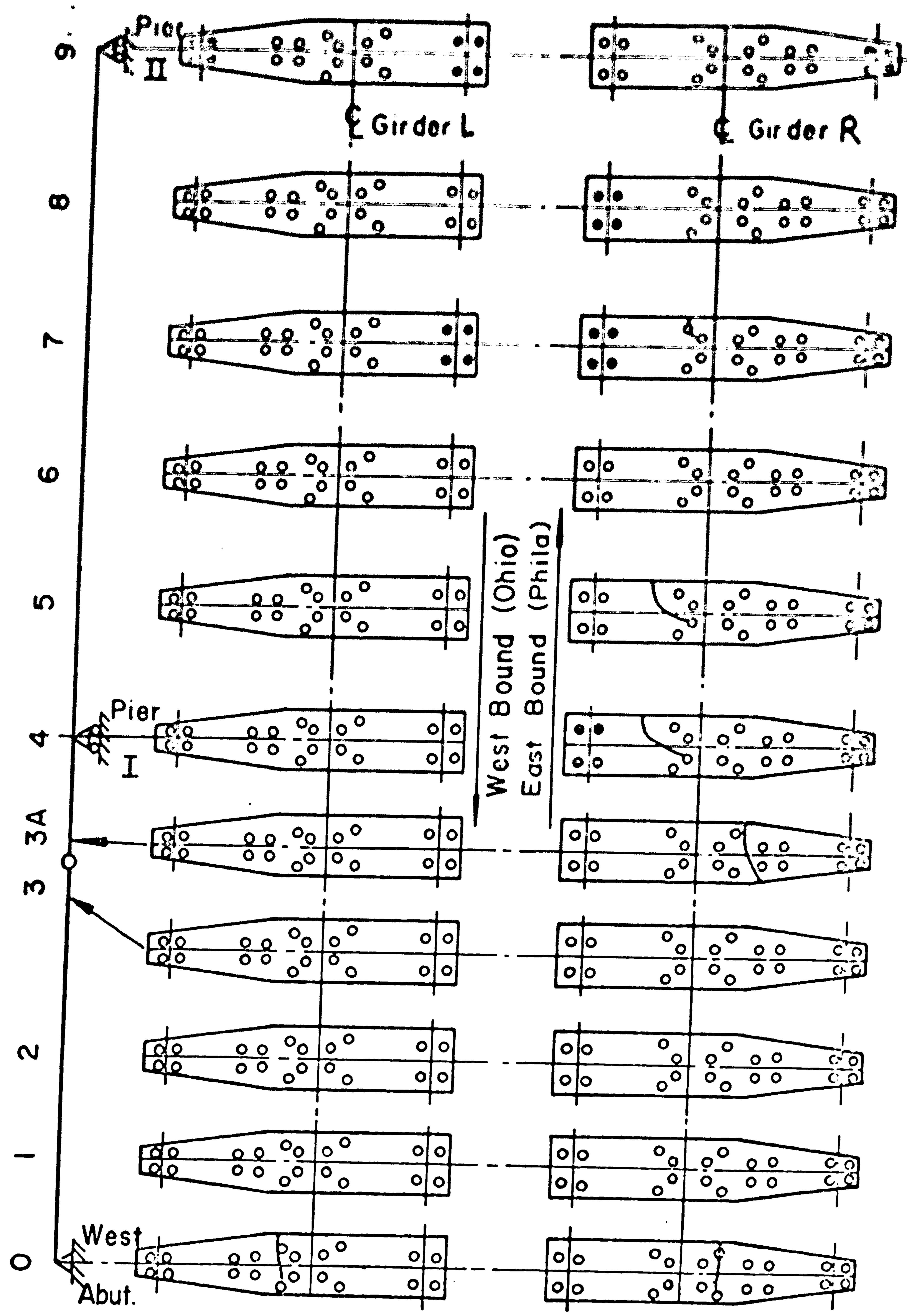


Fig. 5 Schematic of Cracks and Loose Rivets in Tie-Plates of Test Span

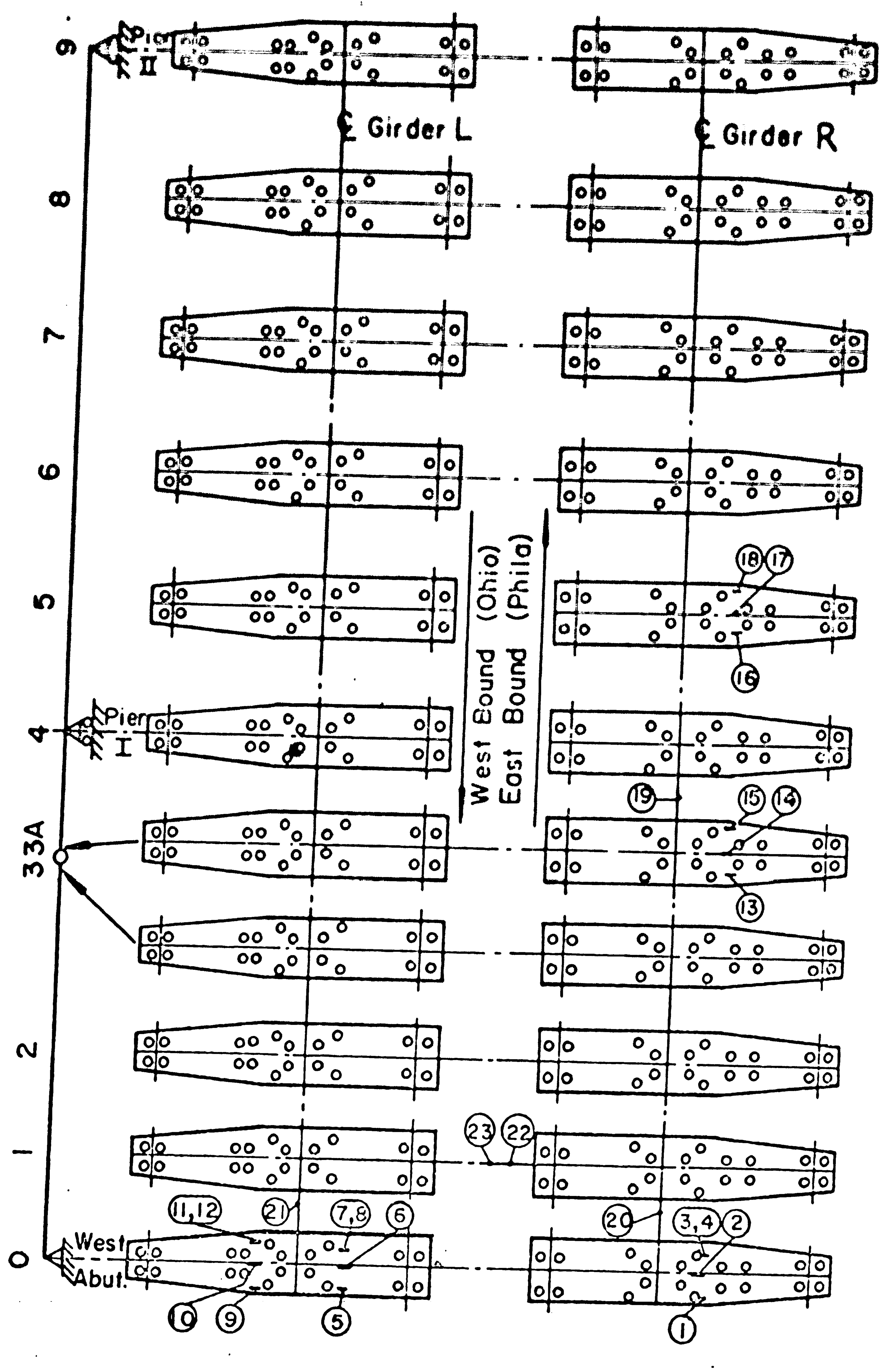


Fig. 6 Strain Gage Location and Identification

Fig. 7 Strain Gages on Tie-Plate

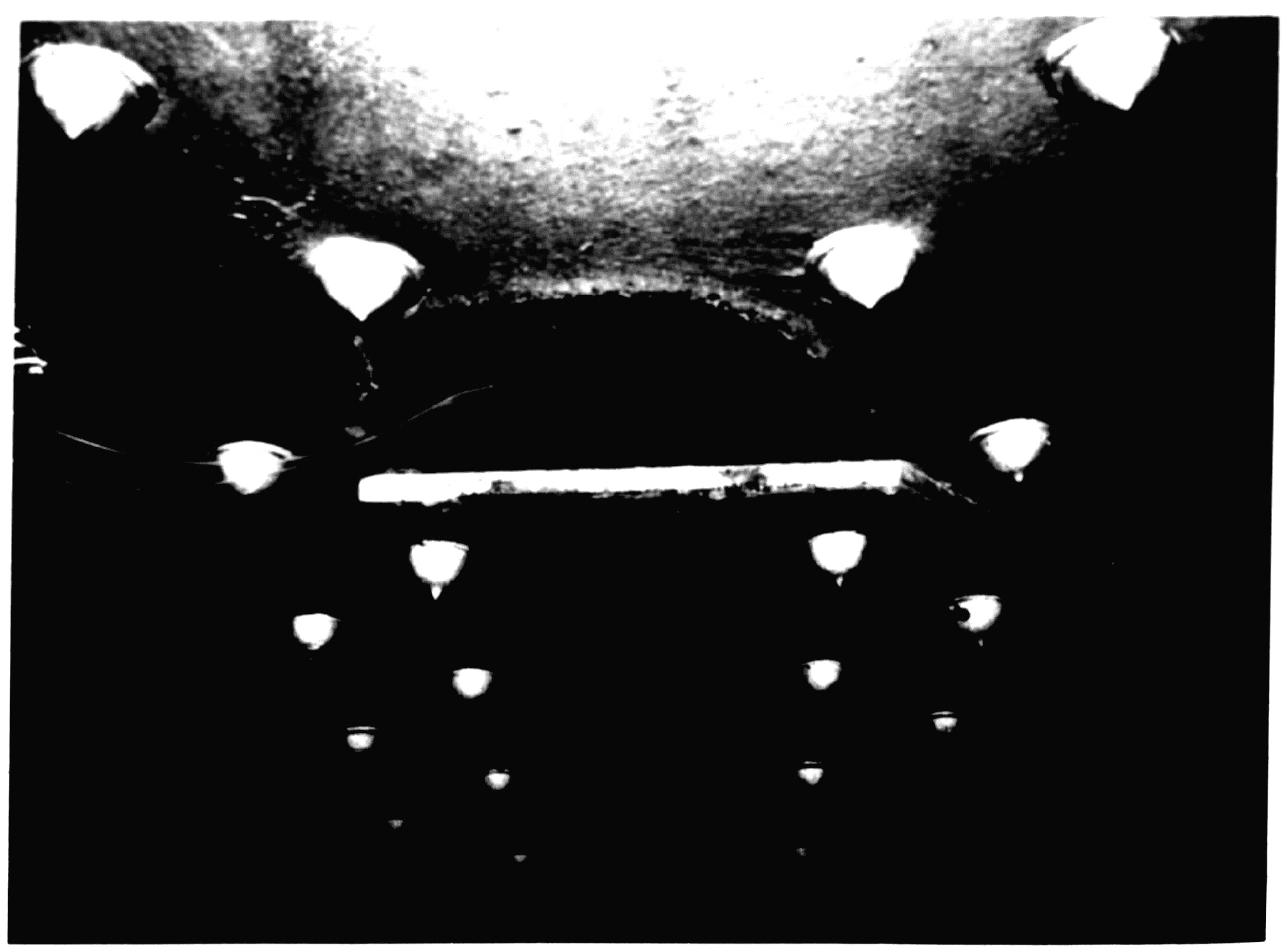


Fig. 8 Strain Gage on Bottom of Girder

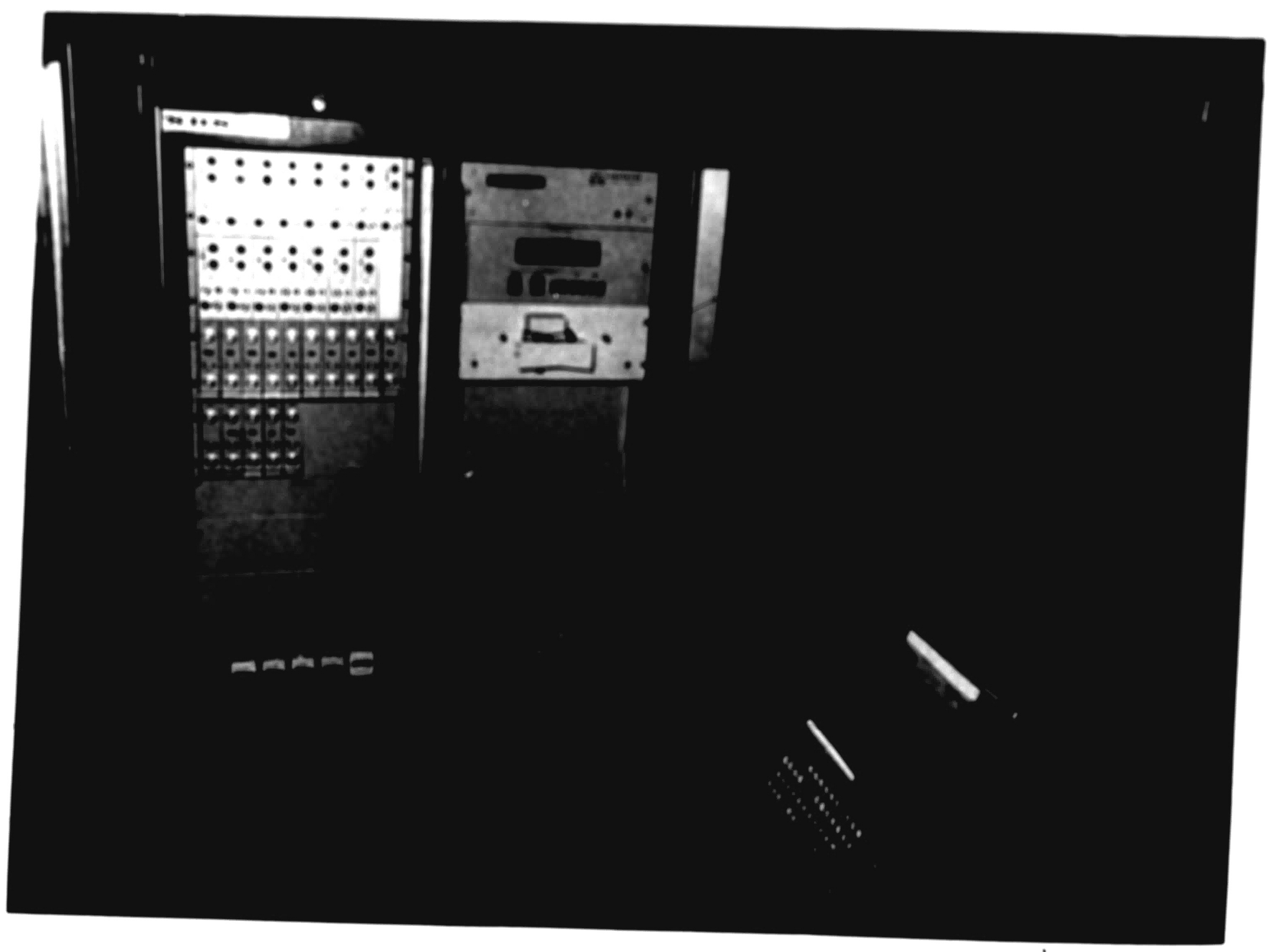


Fig. 9 FHWA Data Acquisition System

STRAIN RANGE	CHANNEL 1	2	3	4	5	6	7	8	9	10
1	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0
3	0	0	0	1	1	1	0	2	2	0
4	0	0	0	6	9	1	2	3	3	1
5	0	0	0	7	7	0	3	4	5	1
6	1	0	0	9	8	0	5	4	5	1
7	3	1	0	12	10	1	7	7	6	3
8	4	11	8	36	31	6	11	30	34	1
9	7	23	145	118	104	108	[illegible]	87	104	65
	1	18	218	55	65	30	214	453	502	284

ZERO LEVEL ADJUSTMENT - 1949,T + 0

CH. 1=	- 21	- 25	- 26	- 29	- 30
CH. 2=	2	0	0	2	4
CH. 3=	4	- 16	- 14	- 19	- 6
CH. 4=	- 118	- 118	- 130	- 139	- 156
CH. 5=	- 45	- 32	- 35	- 32	- 44
CH. 6=	- 31	- 23	- 26	- 31	- 44
CH. 7=	- 16	- 21	- 27	- 24	- 16
CH. 8=	5	- 5	- 4	- 3	12
CH. 9=	- 19	- 31	- 31	- 29	- 11
CH. 10=	- 74	- 61	- 66	- 74	- 91

Fig. 10 Typical Data Printout

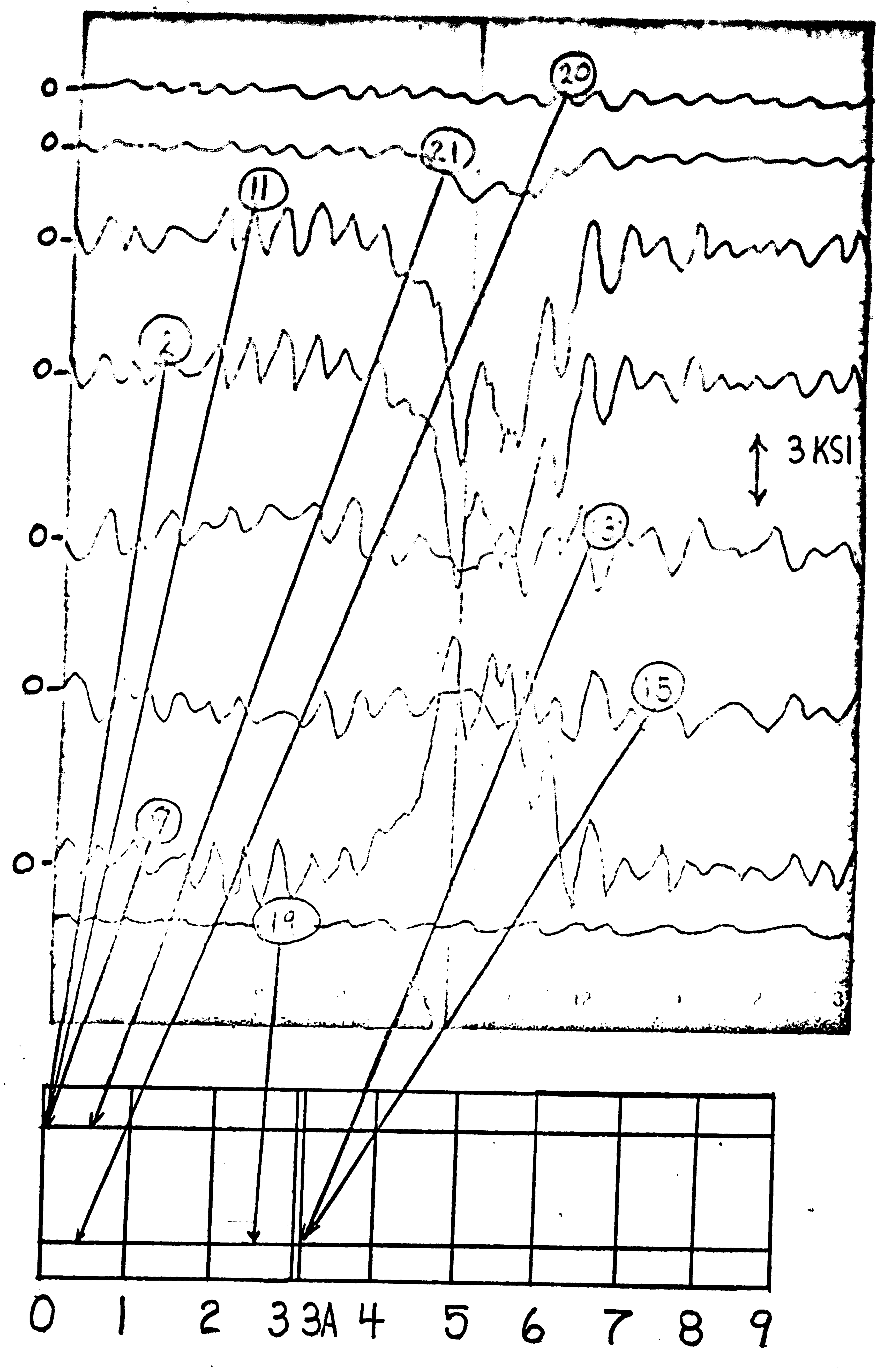


Fig. 11 Typical Response of Gages Attached to Tie-Plates and Girder

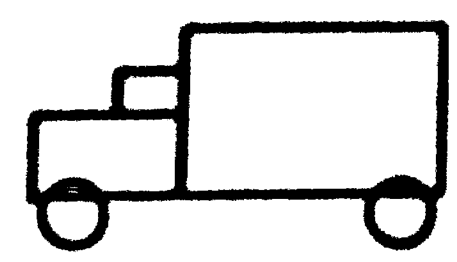
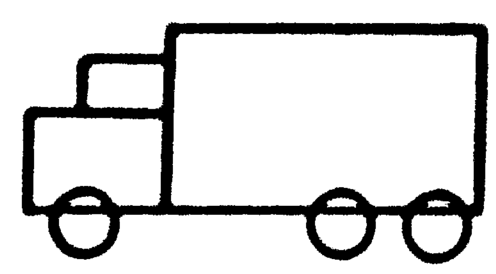
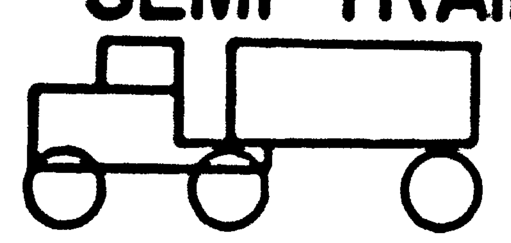
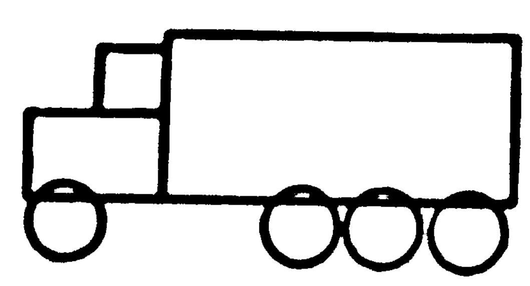
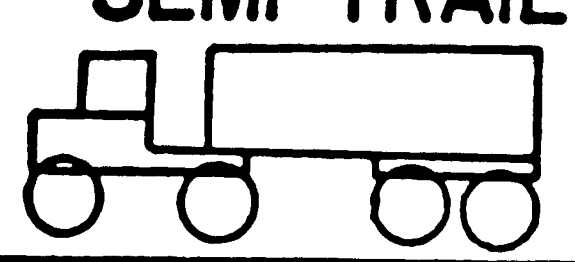
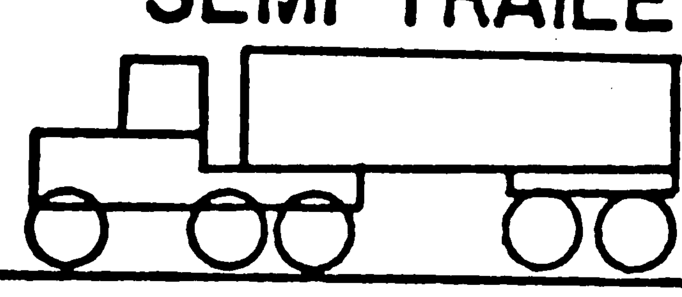
TYPE	CODE
BUS	B
2-AXLE TRUCK 	2D
3-AXLE TRUCK 	3
3-AXLE TRACTOR SEMI-TRAILER 	2S-1
4-AXLE TRUCK 	4
4-AXLE TRACTOR SEMI-TRAILER 	2S-2
5-AXLE TRACTOR SEMI-TRAILER 	3S-2

Fig. 12 Truck Classification (FHWA)

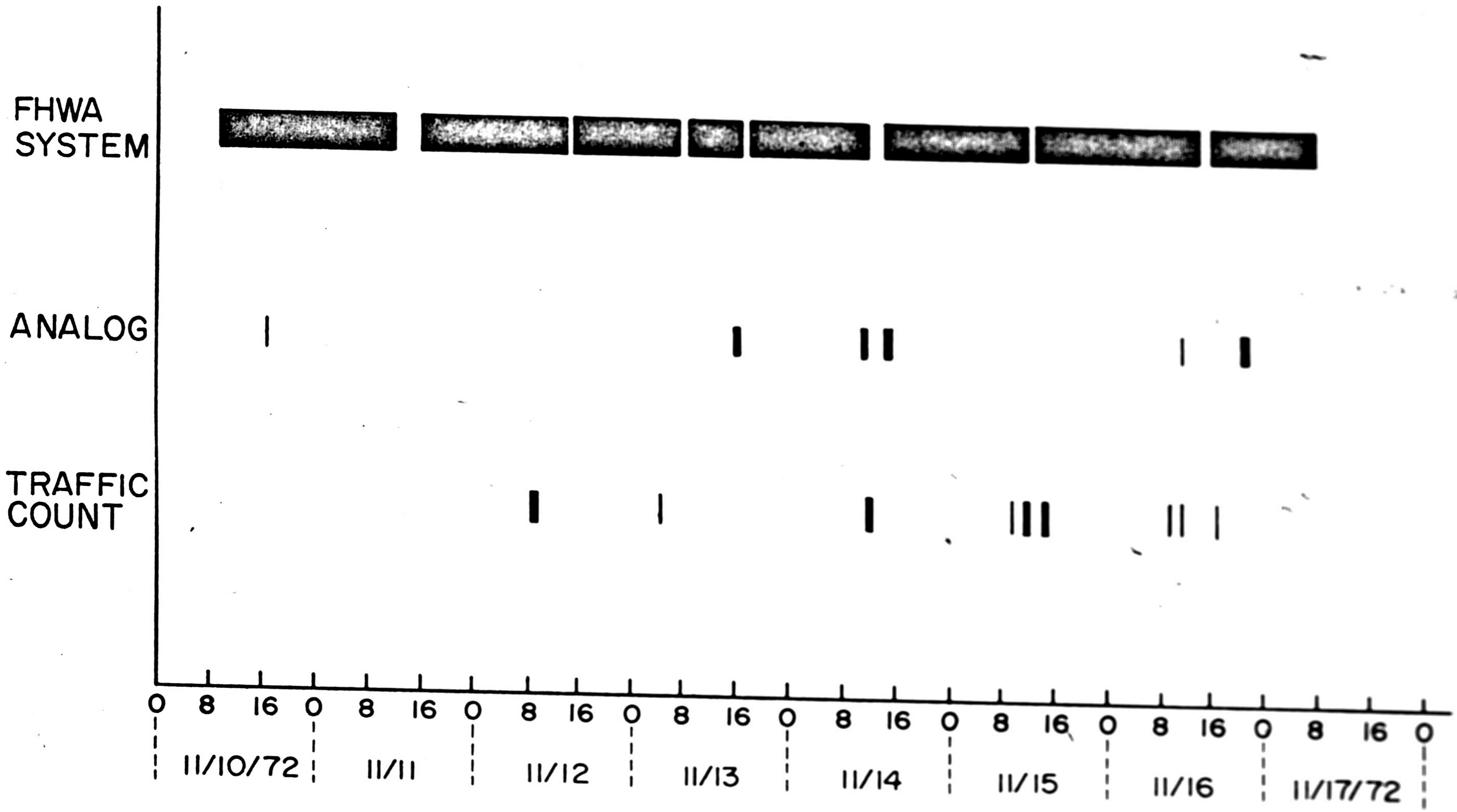


Fig. 13 Chronological Record of Data Acquisition

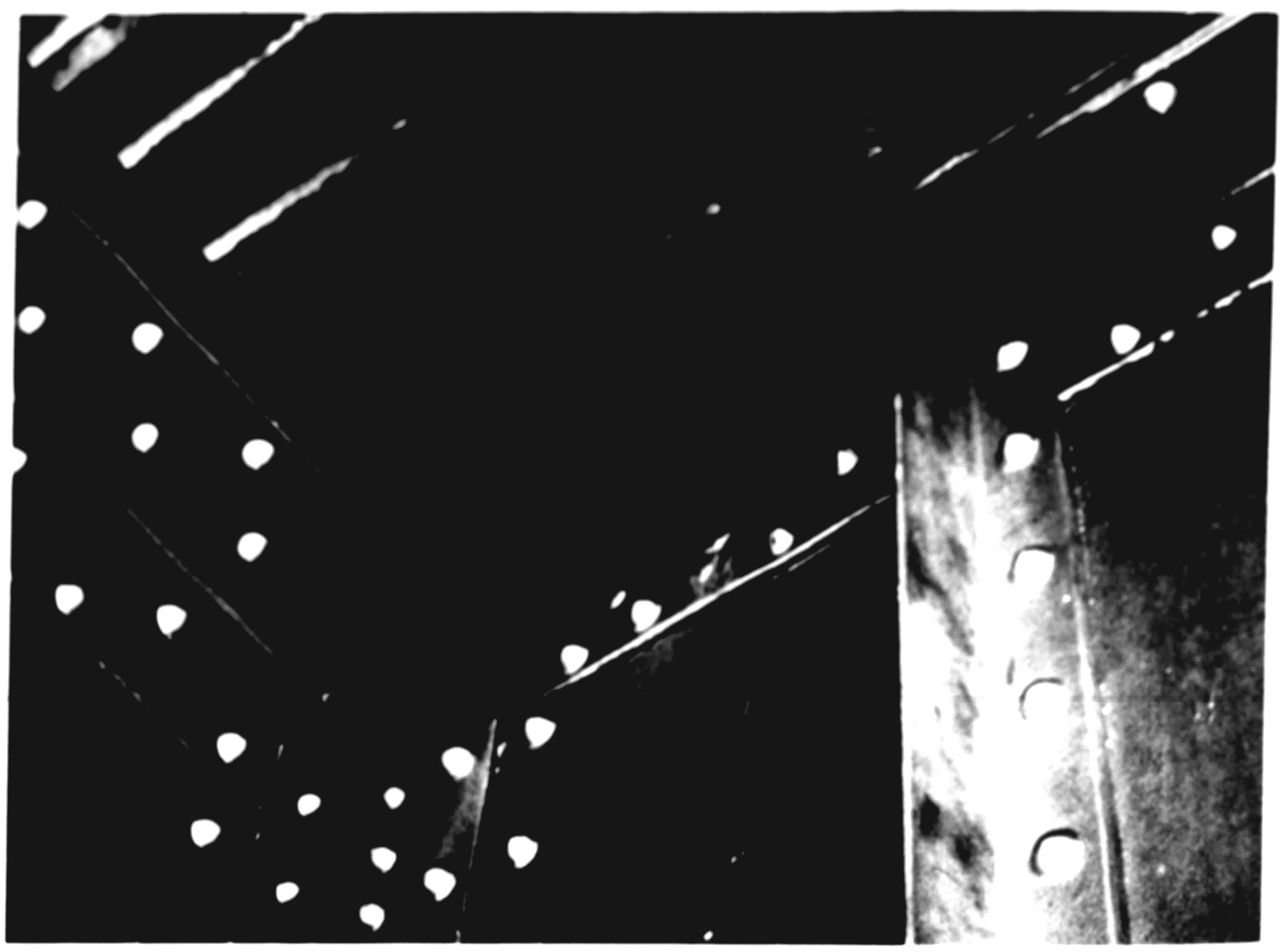


Fig. 14 Reinforcement and Original Tie-Plates (Bottom View)

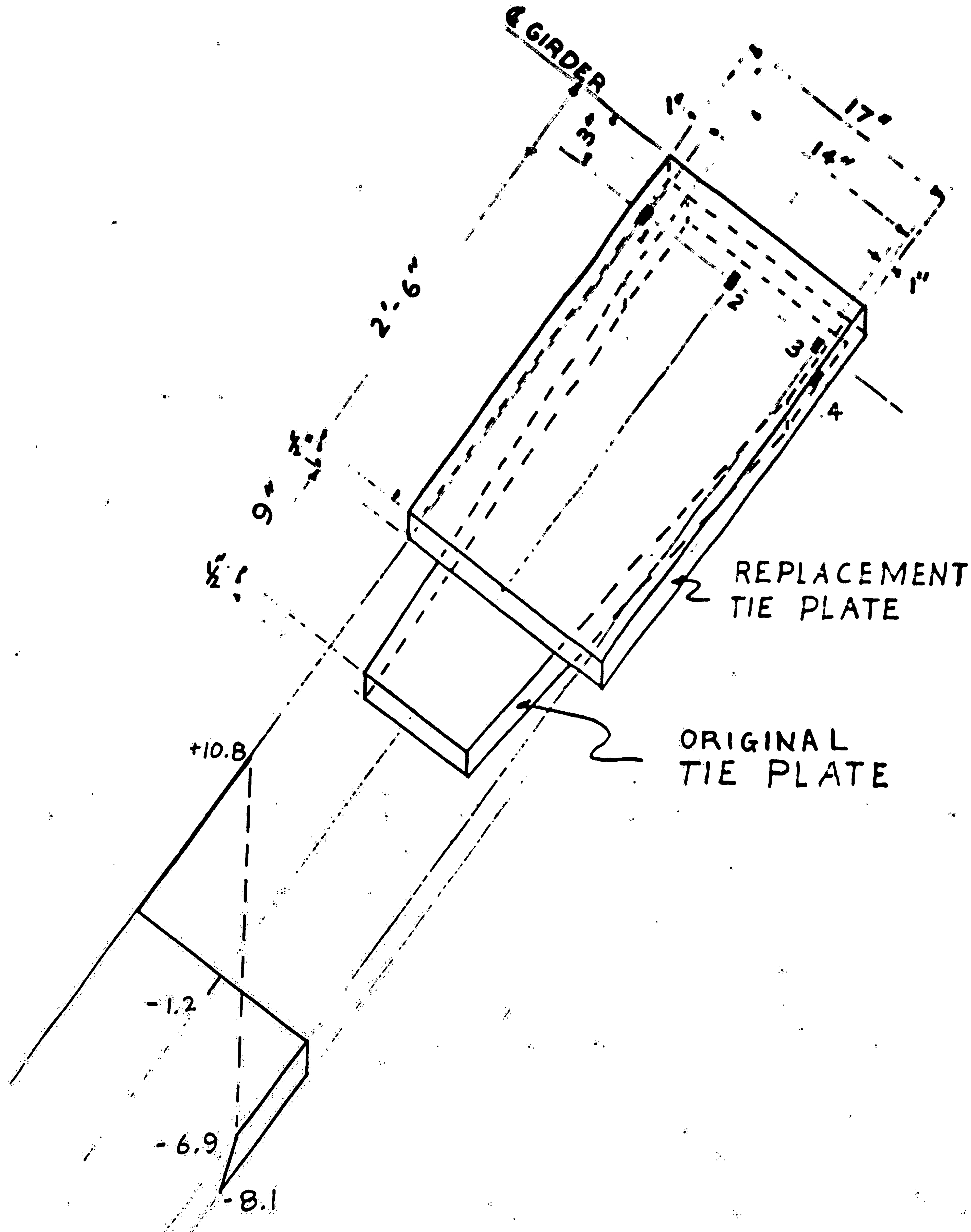


Fig. 15 Stress Distribution in Tie-Plate R-0

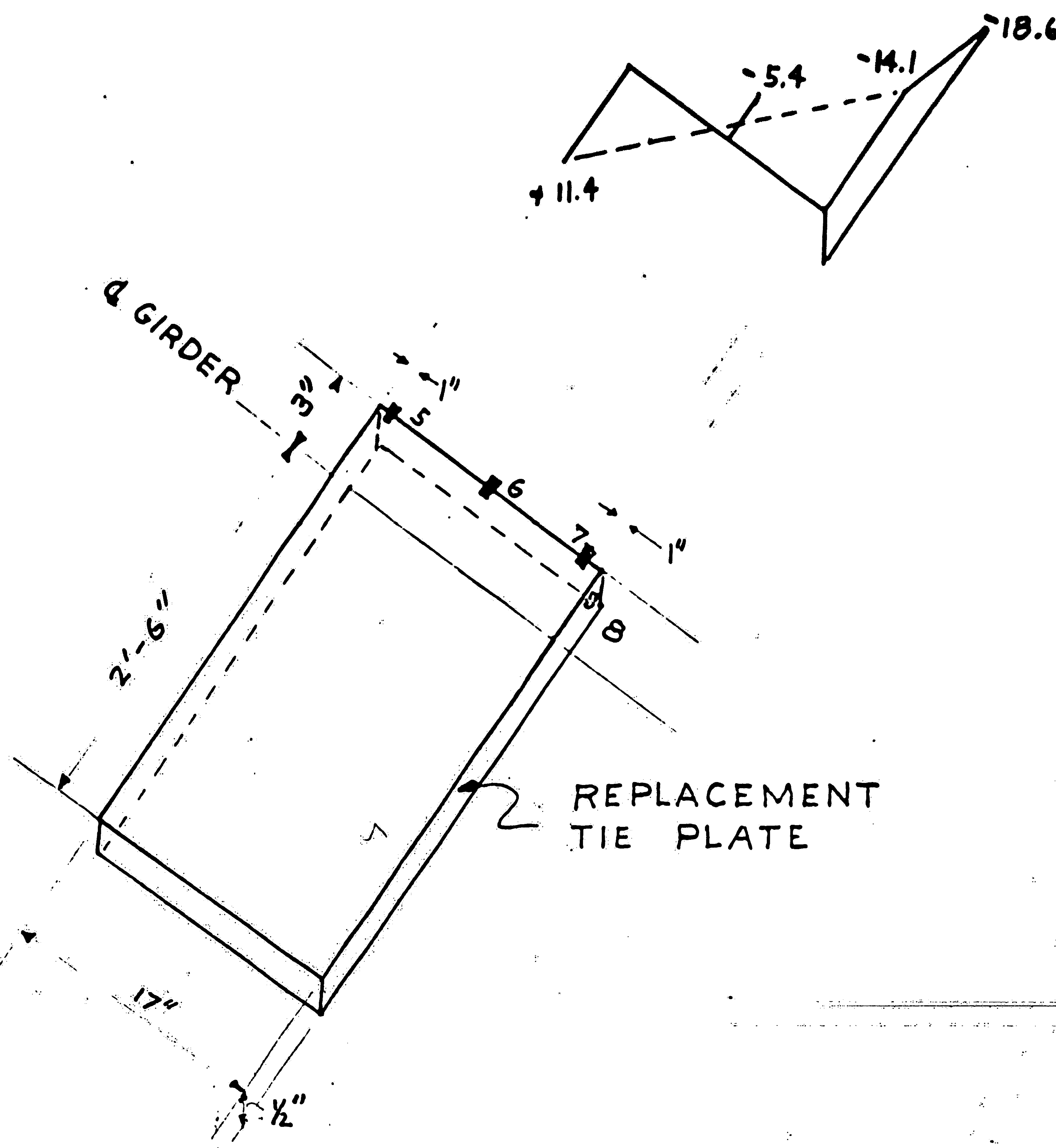


Fig. 16 Stress Distribution in Tie-Plate L-0 Inboard

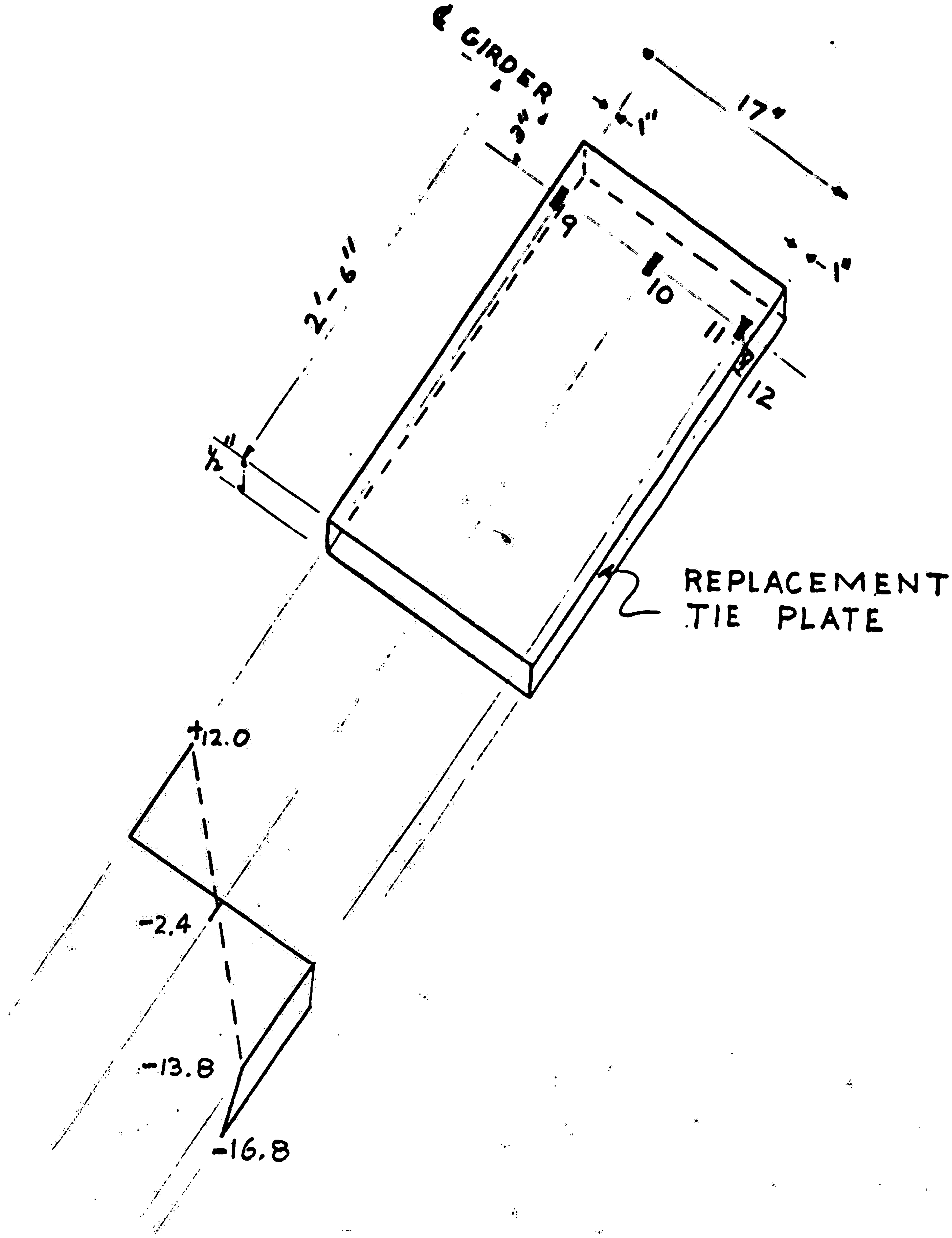


Fig. 17 Stress Distribution in Tie-Plate L-0 Outboard

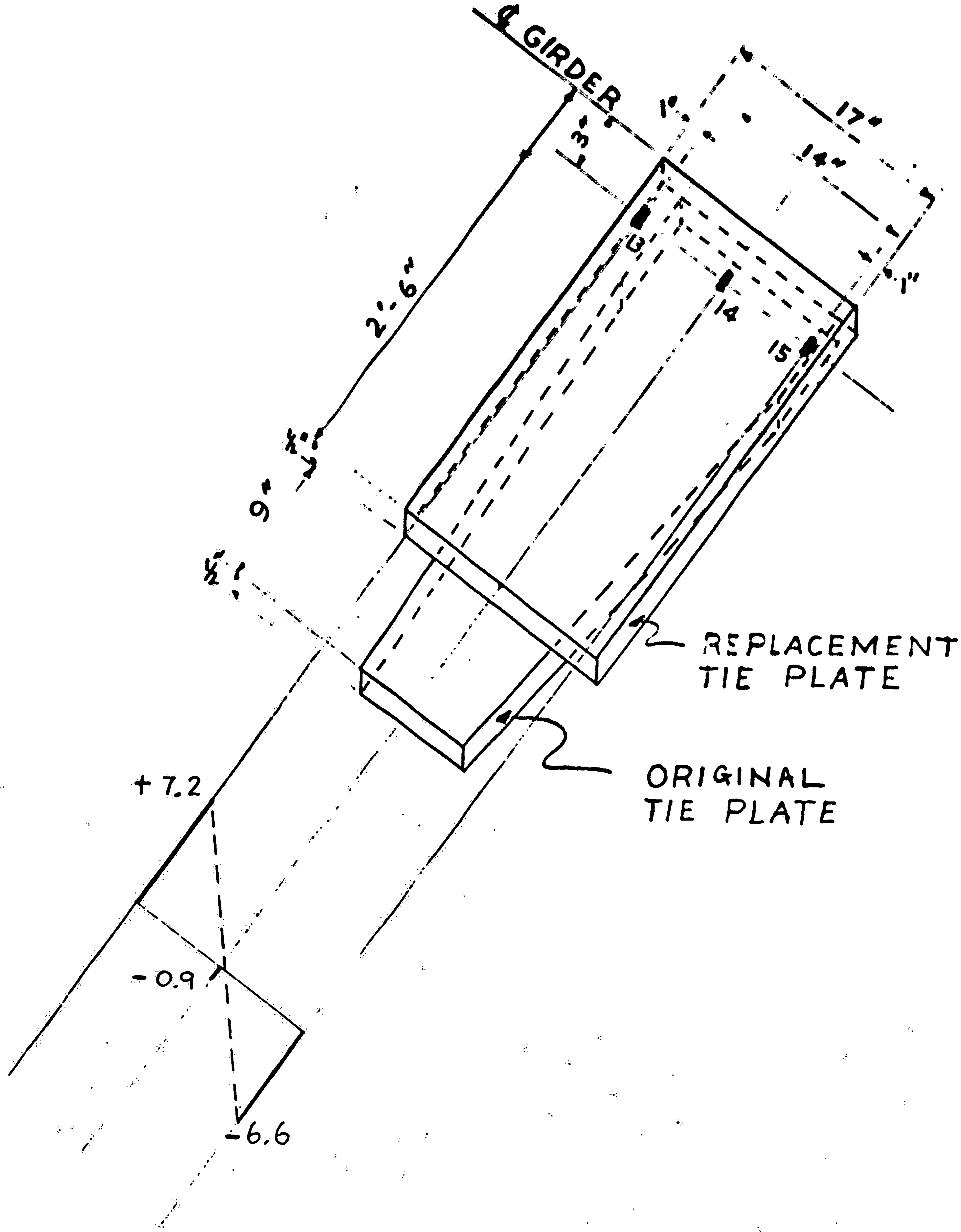


Fig. 18 Stress Distribution in Tie-Plate R-3A

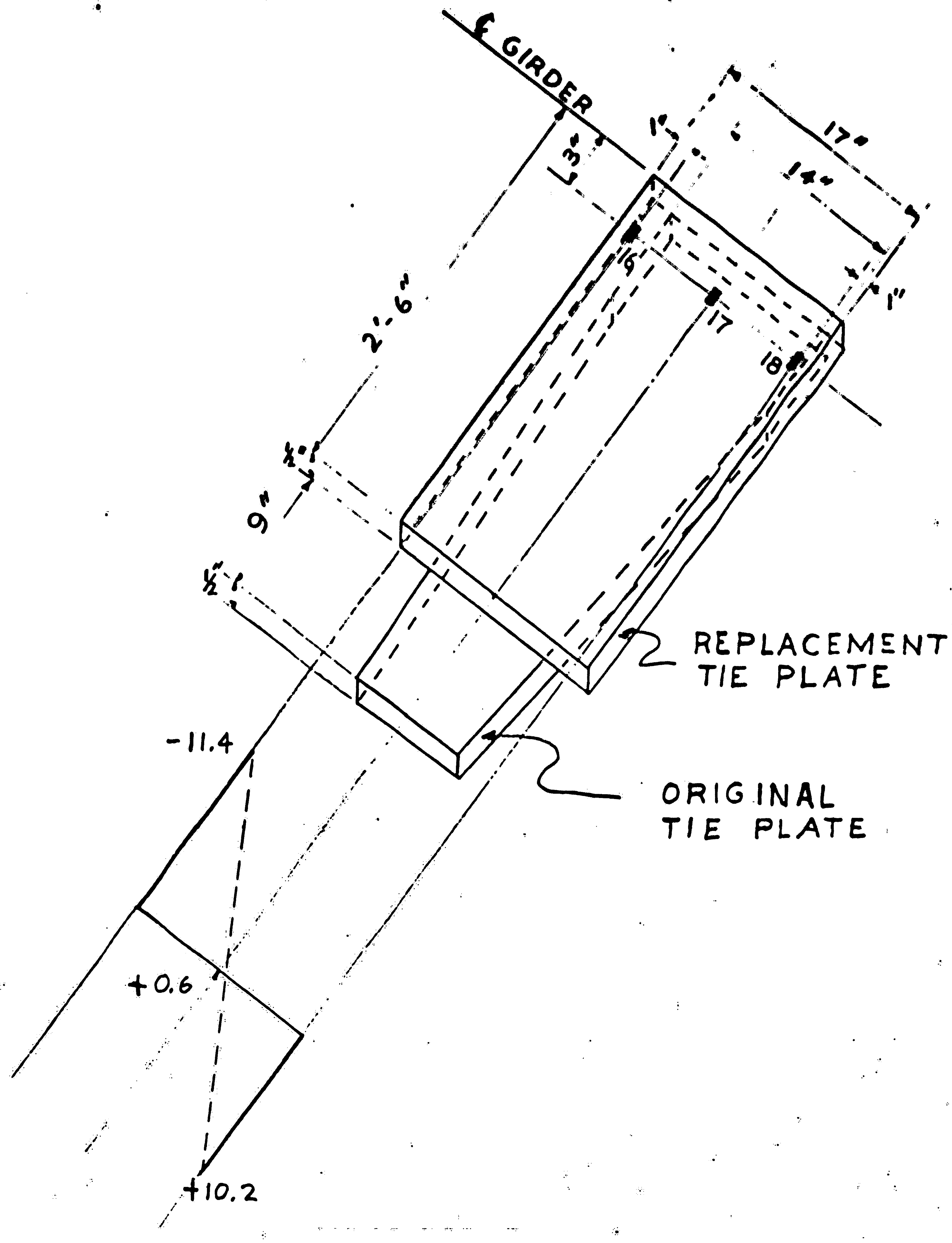


Fig. 19 Stress Distribution in Tie-Plate R-5

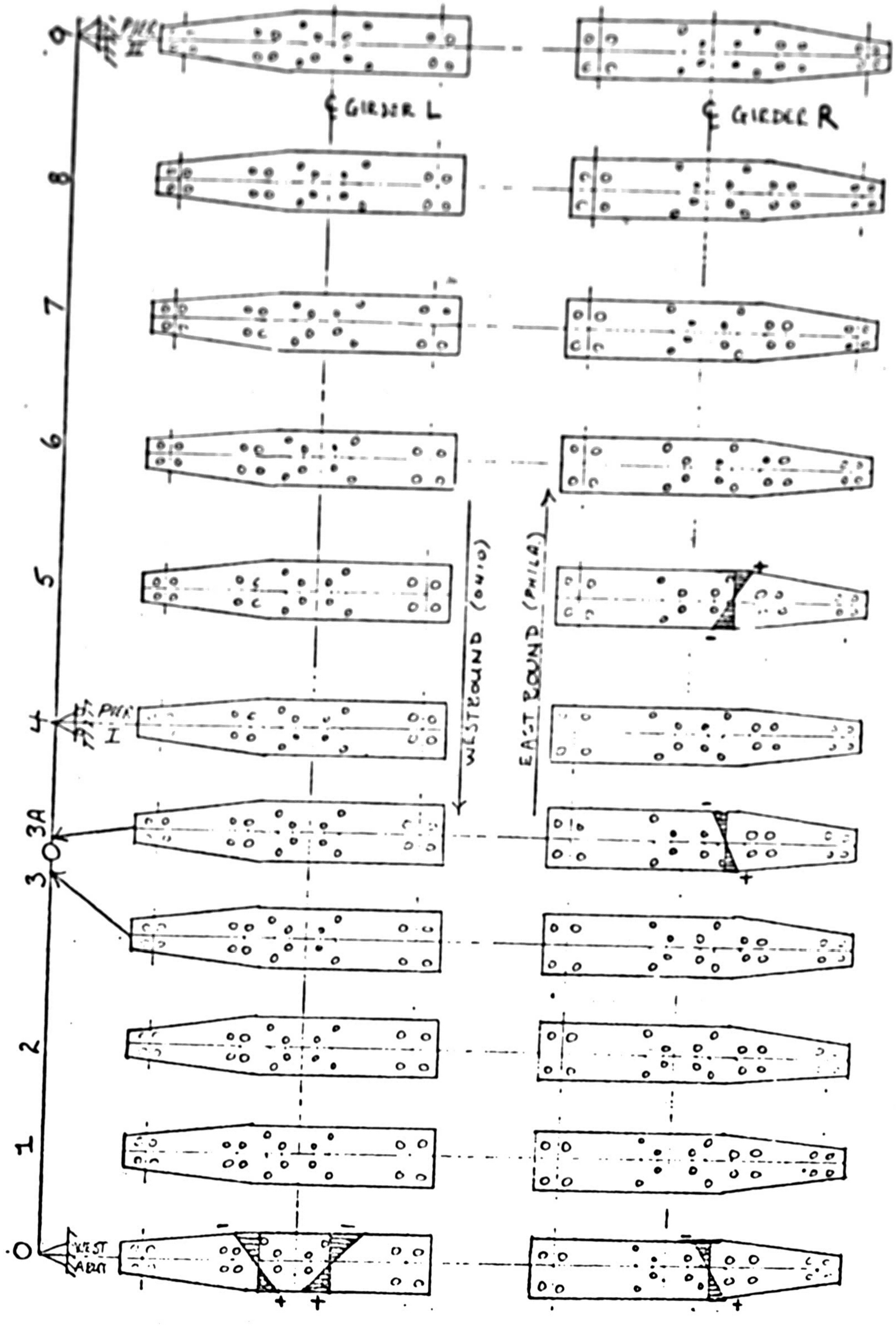


Fig. 20 Instantaneous Stress Pattern in Gaged Tie-Plates

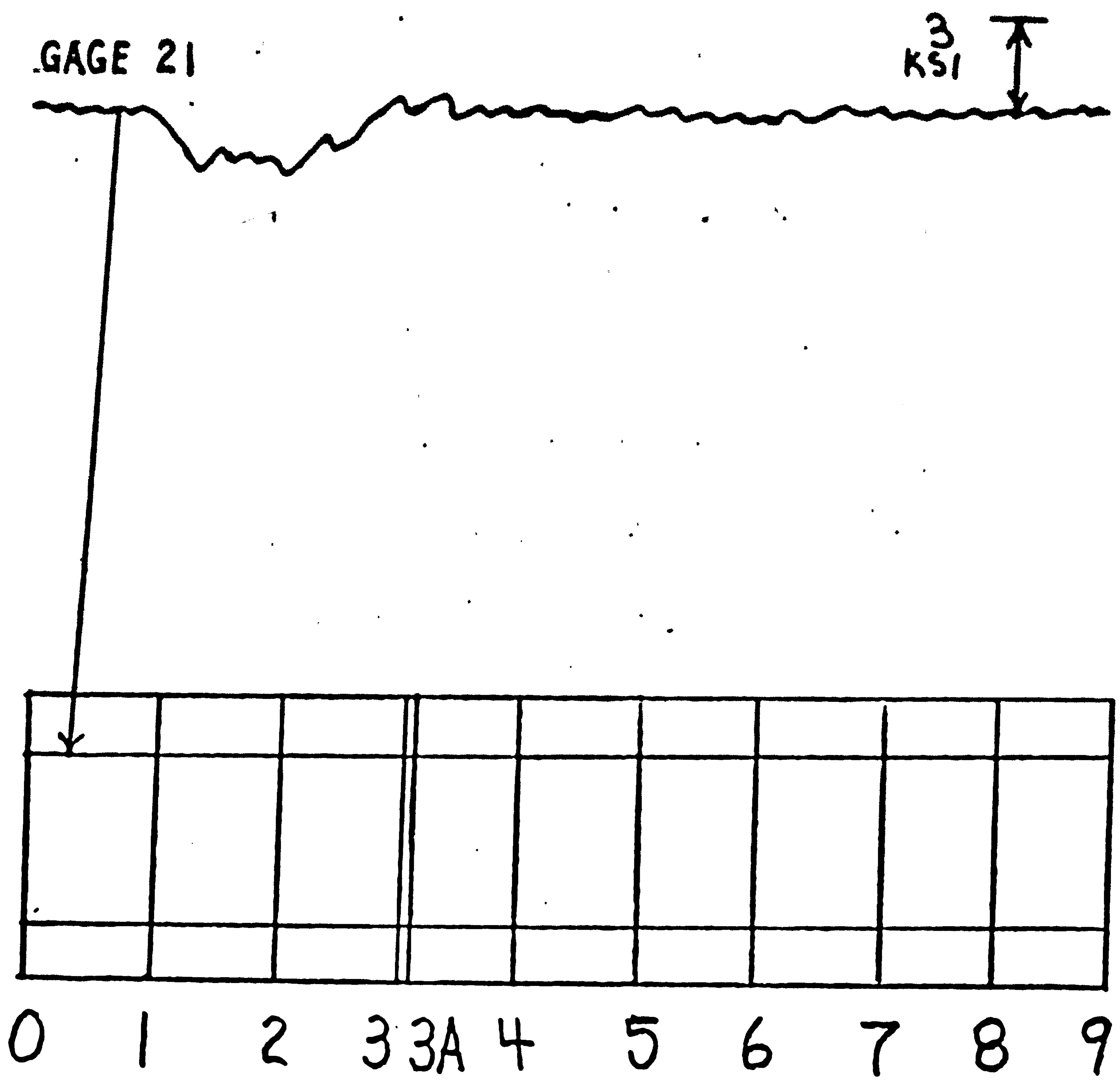


Fig. 21 Strain Variation of a Point on the Girder

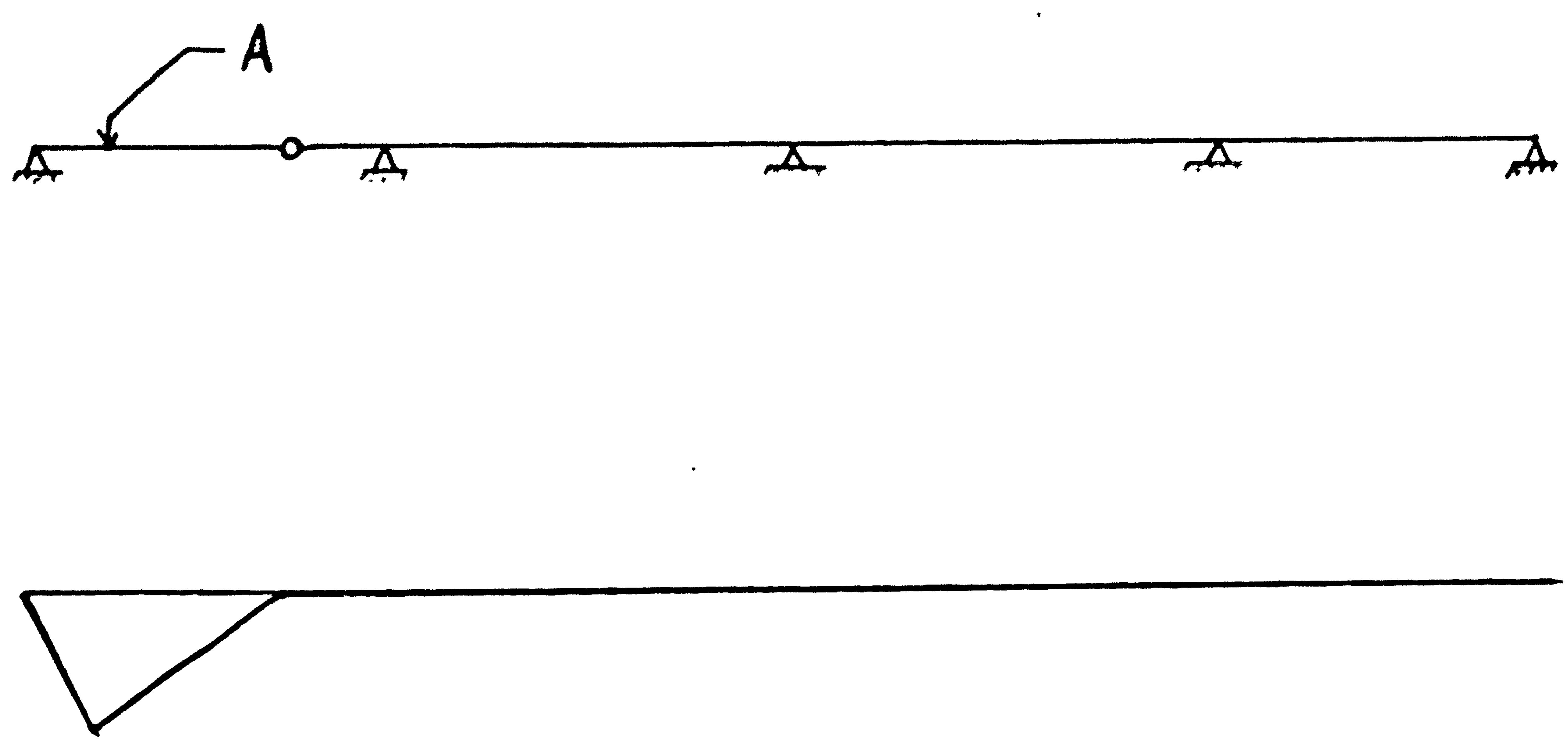


Fig. 22 Schematic of Influence Line for Stress at a Point on the Top Flange of the Girder

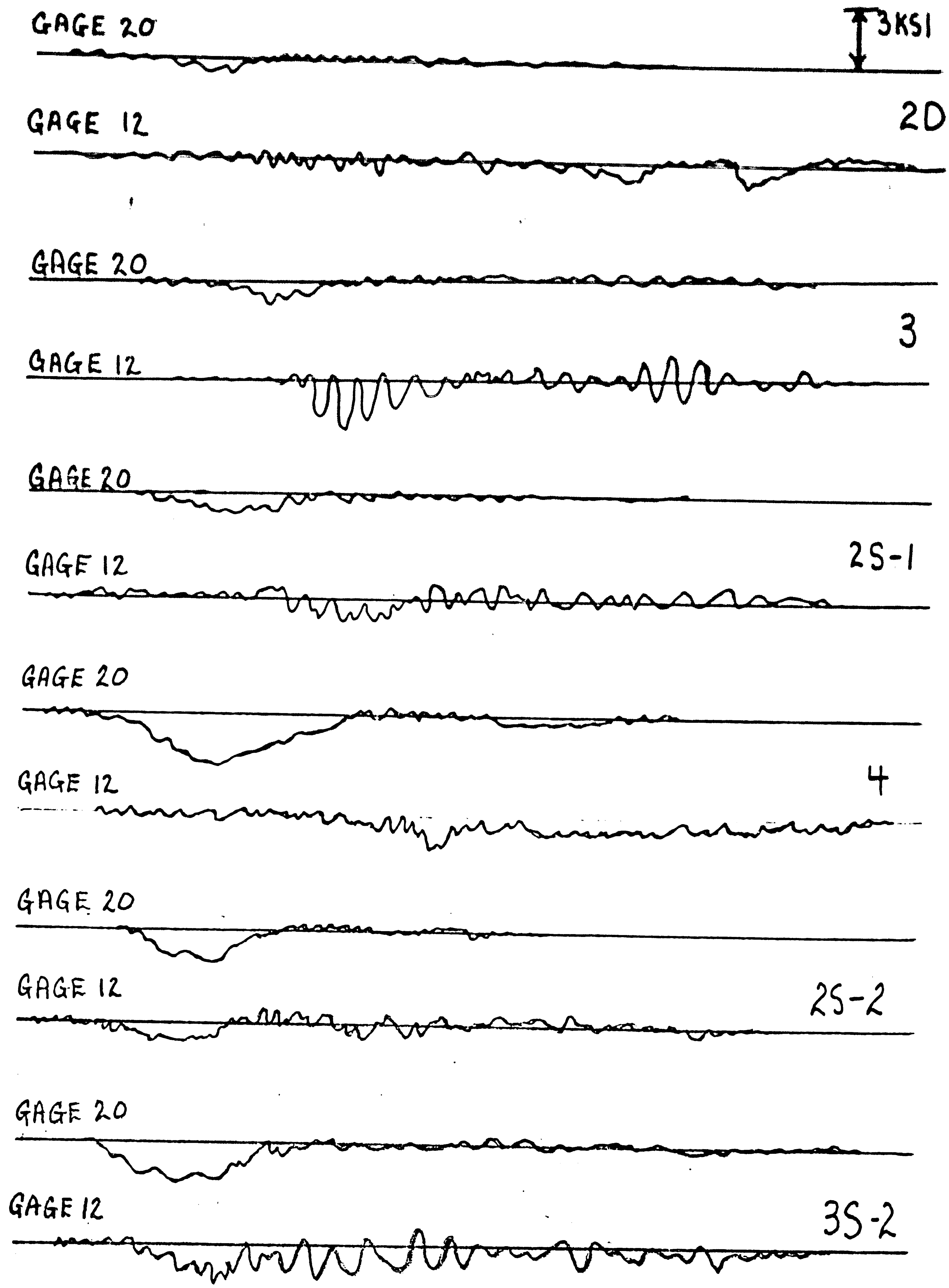


Fig. 23 Typical Stress Response of Tie-Plate and Girder to Truck Traffic

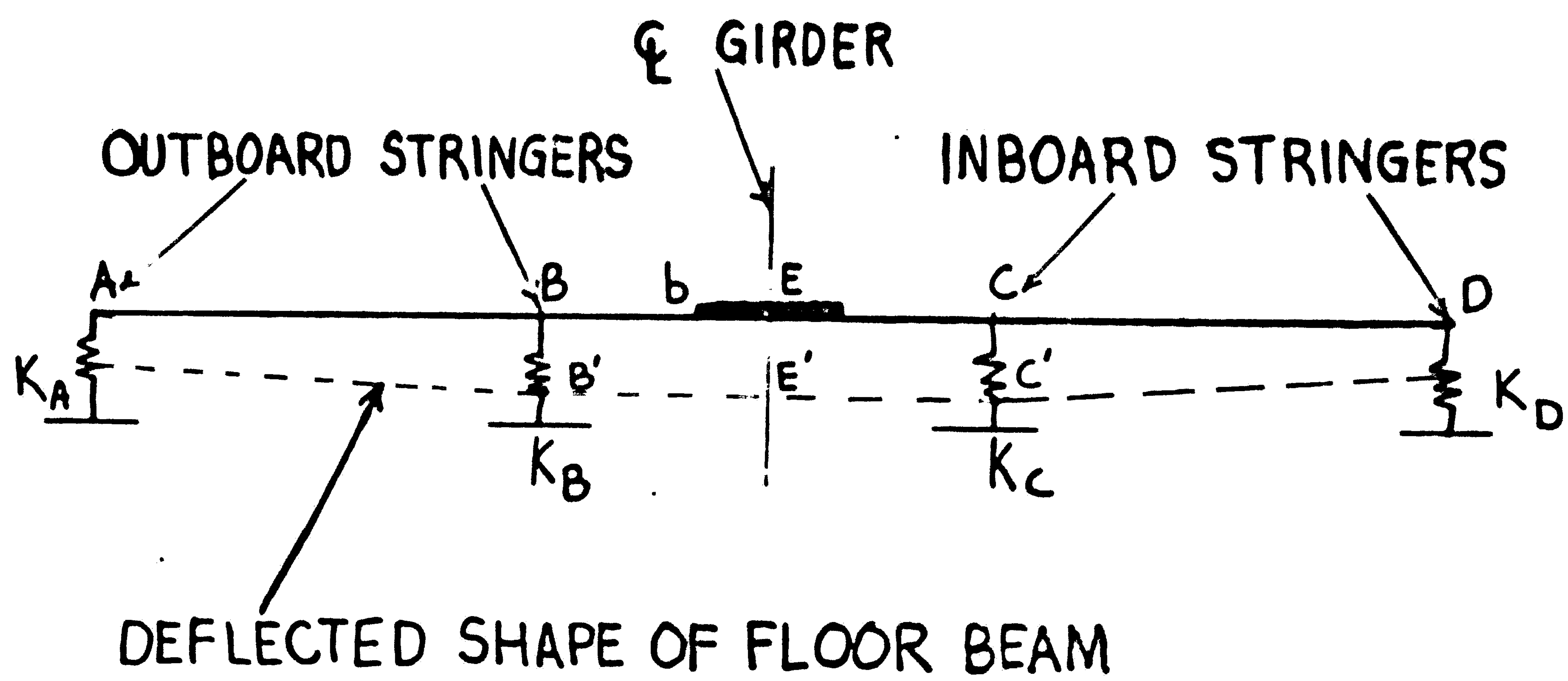


Fig. 24 Assumed Model for Displacement in Tie-Plates

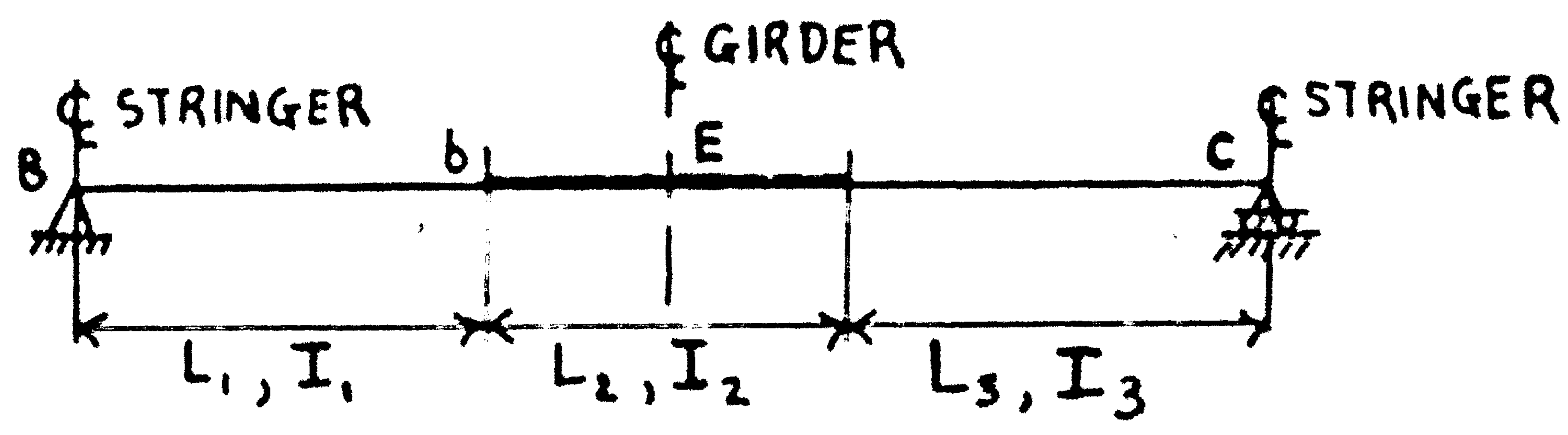


Fig. 25a Boundary Conditions of Model:
Simply Supported

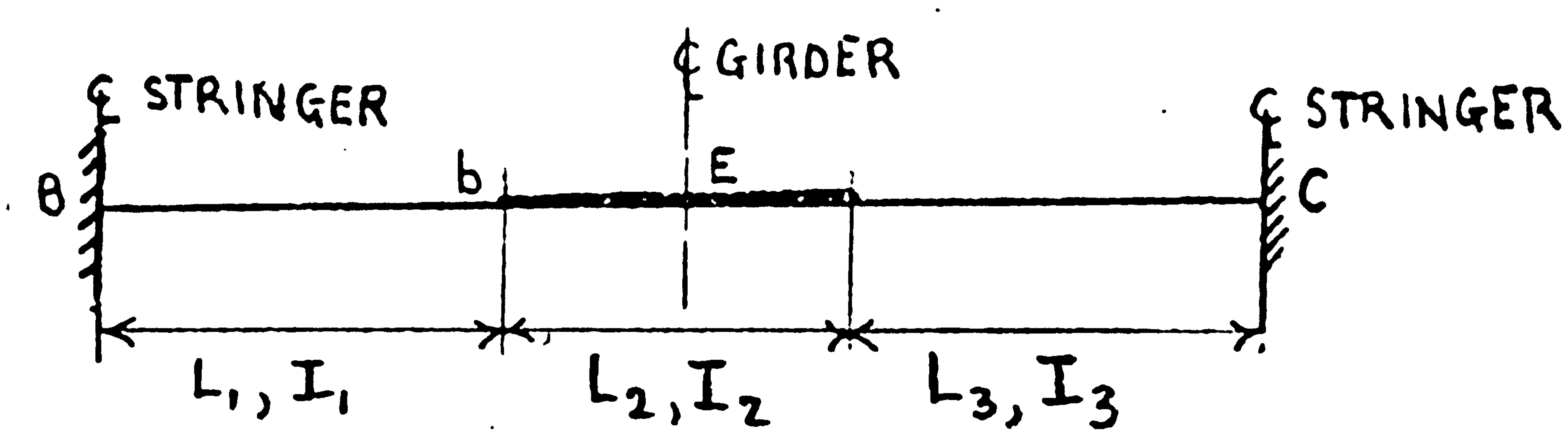


Fig. 25b Boundary Conditions of Model:
Fixed-Fixed

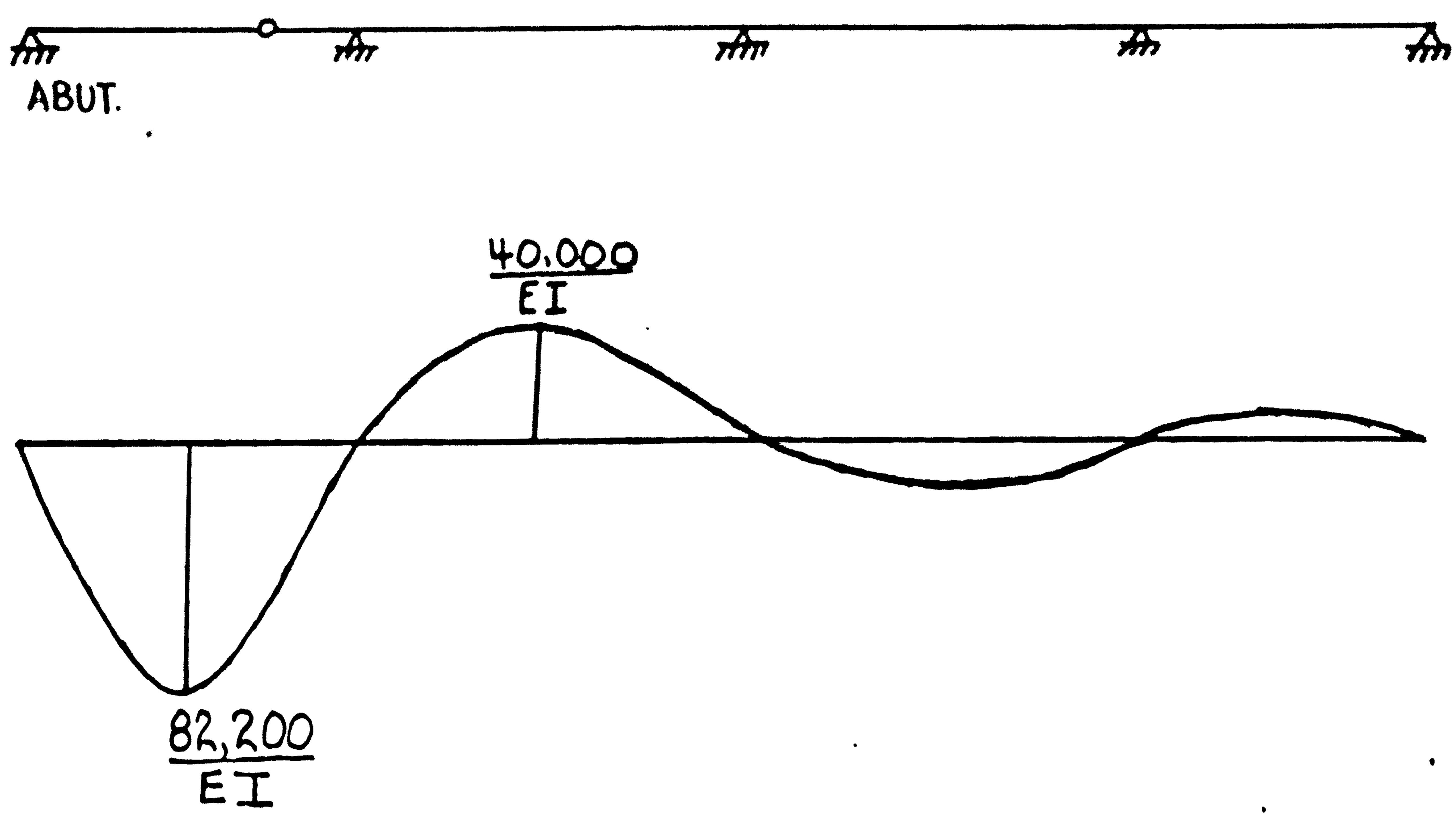
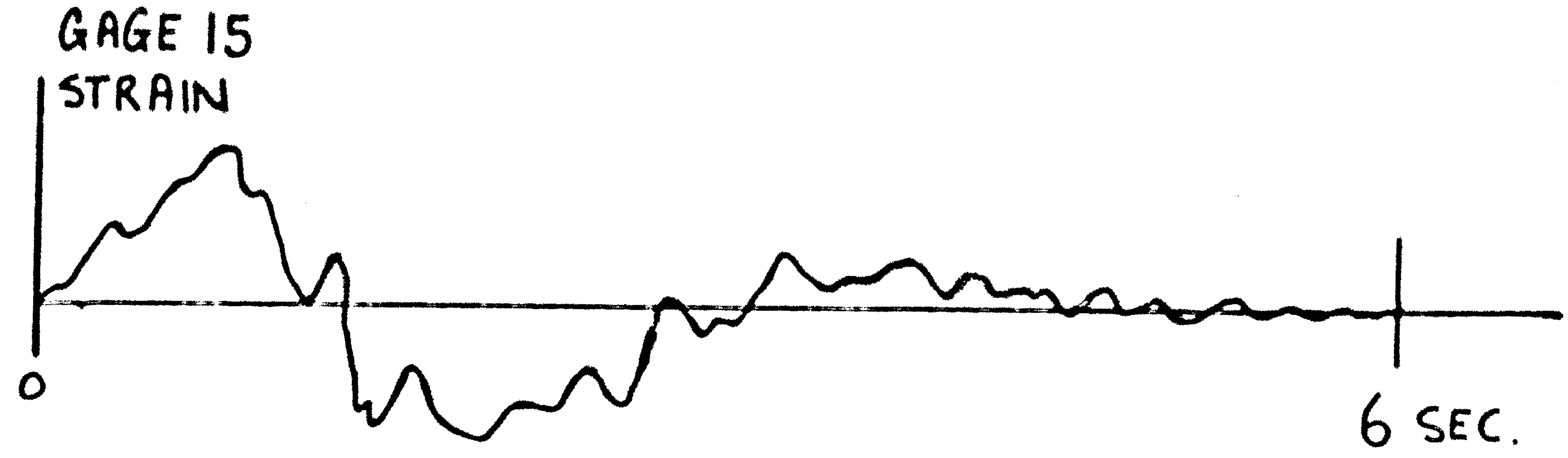


Fig. 26 Influence Line for Girder Slope at the Abutment



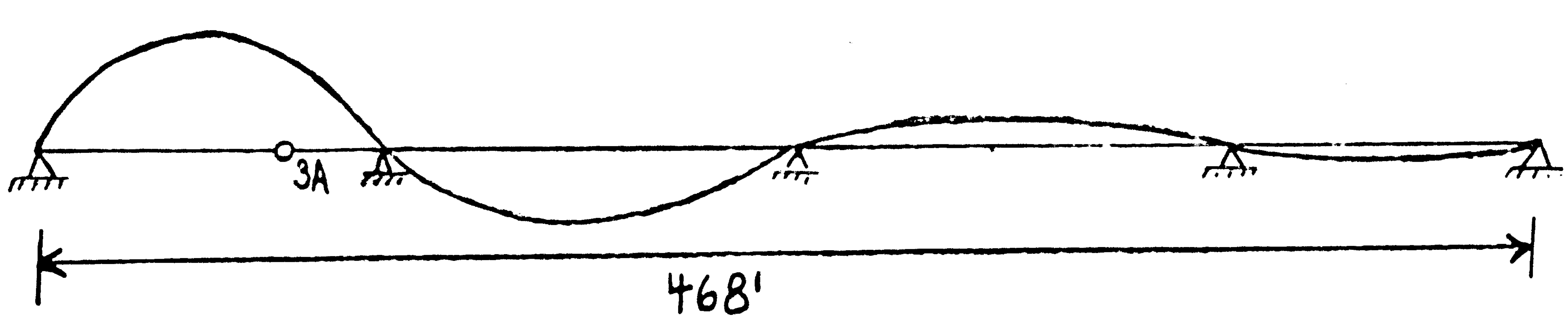


Fig. 27 Comparison of Strain Variation in Tie-Plate and Schematic of Influence Line for Girder Slope at 3A-R

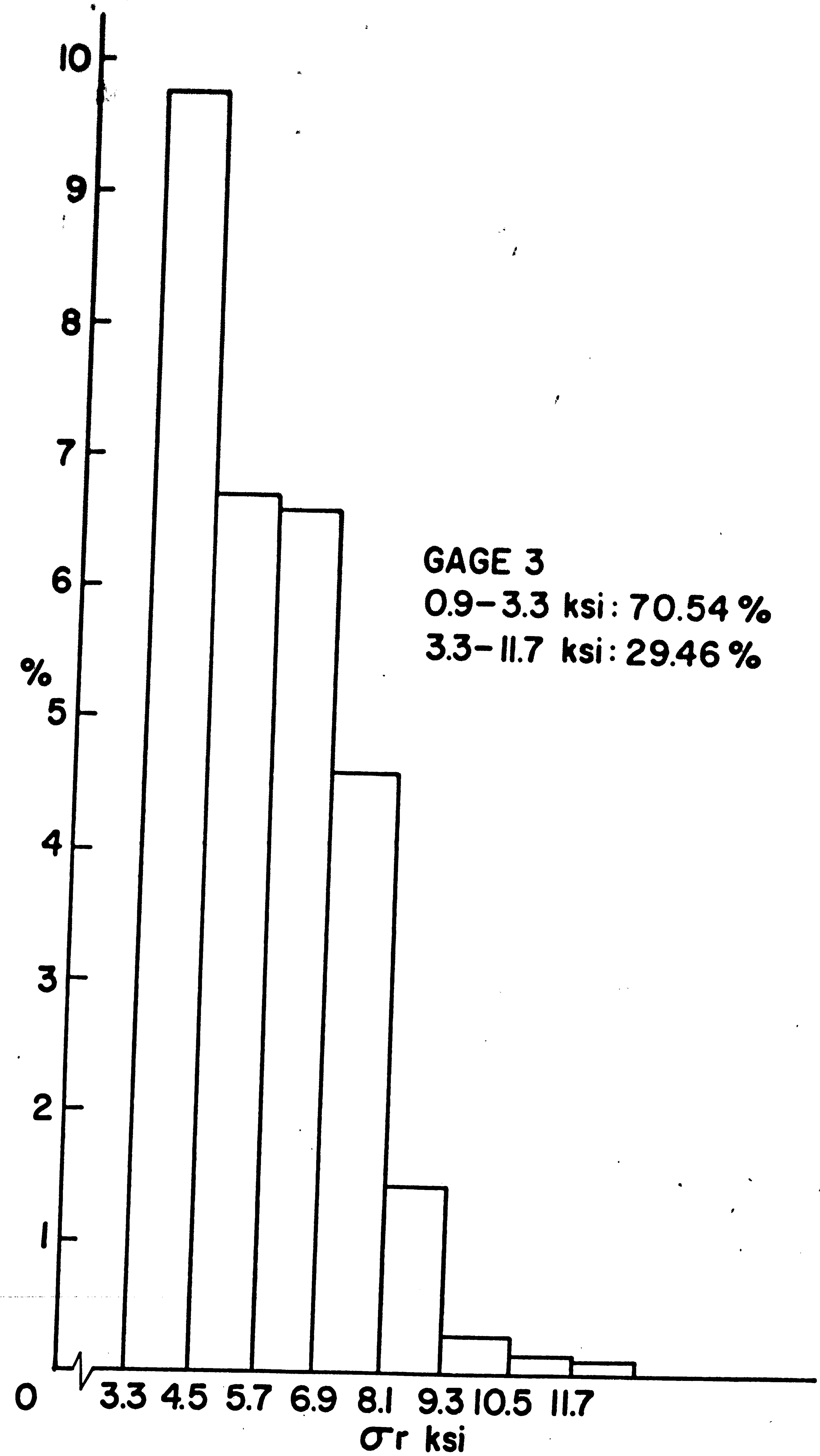


Fig. 28 Histogram for Gage 3 (Tie-Plate RT-0)

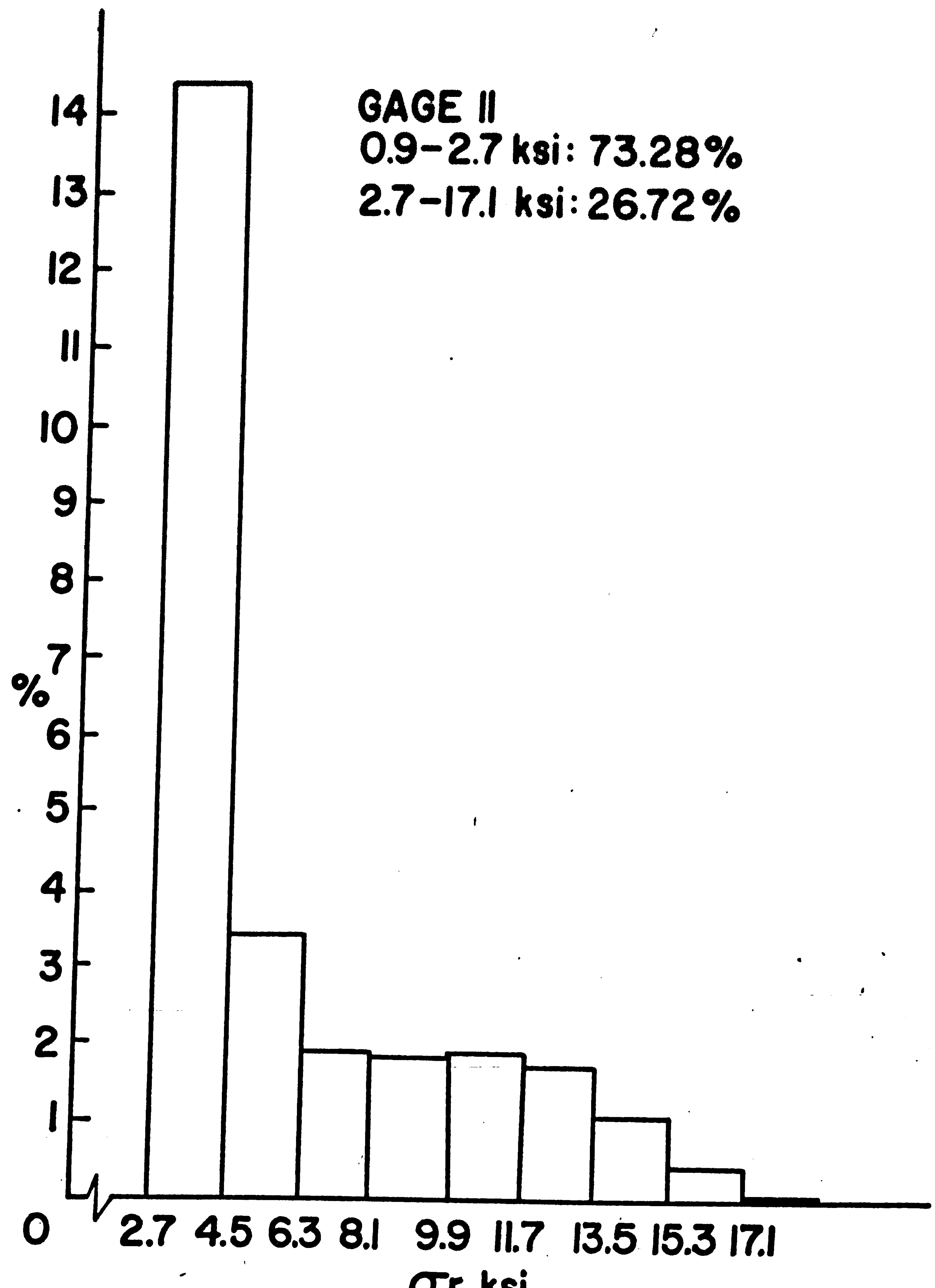


Fig. 29 Histogram for Gage 11 (Tie-Plate LT-0)

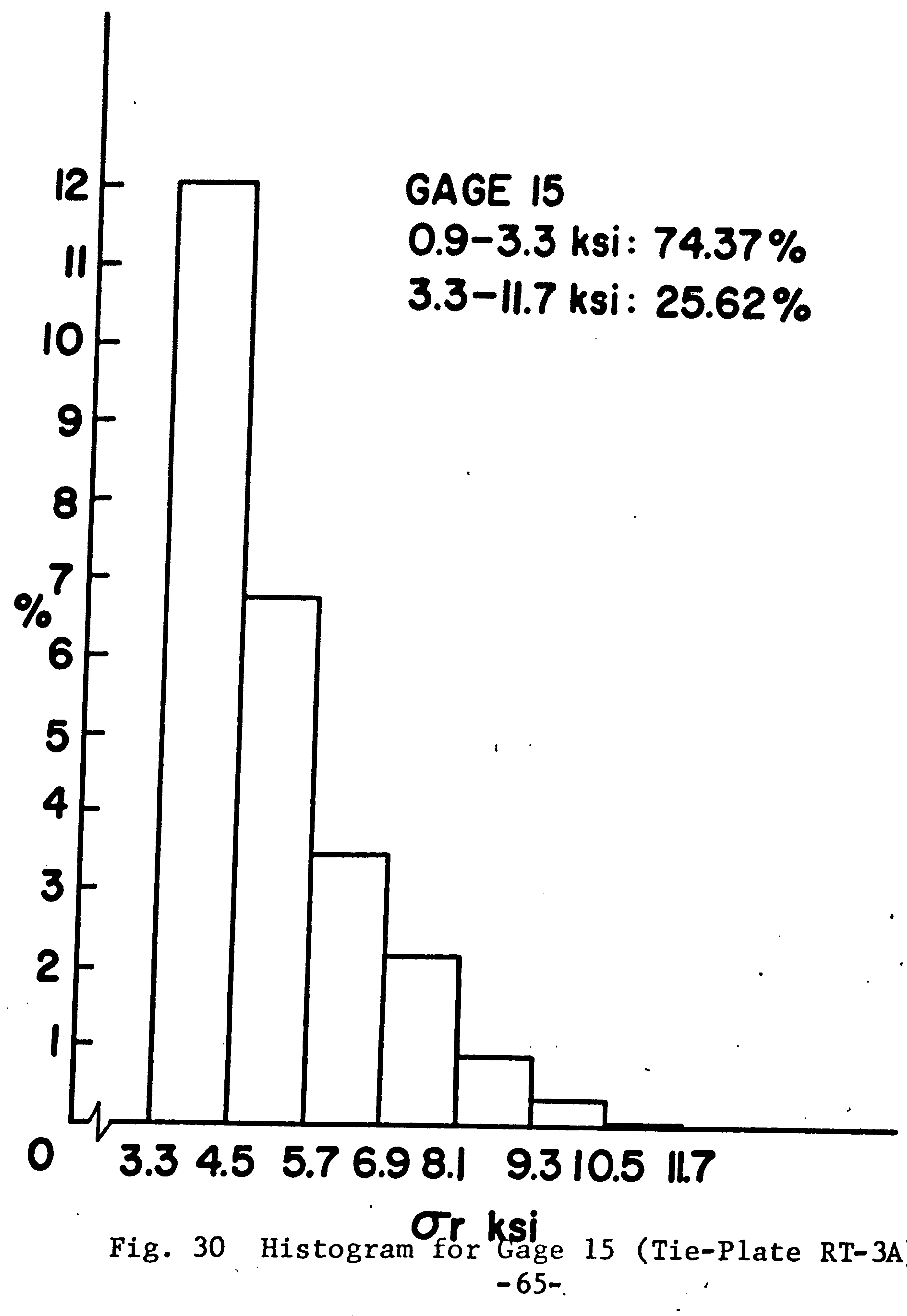


Fig. 30 Histogram for Gage 15 (Tie-Plate RT-3A)

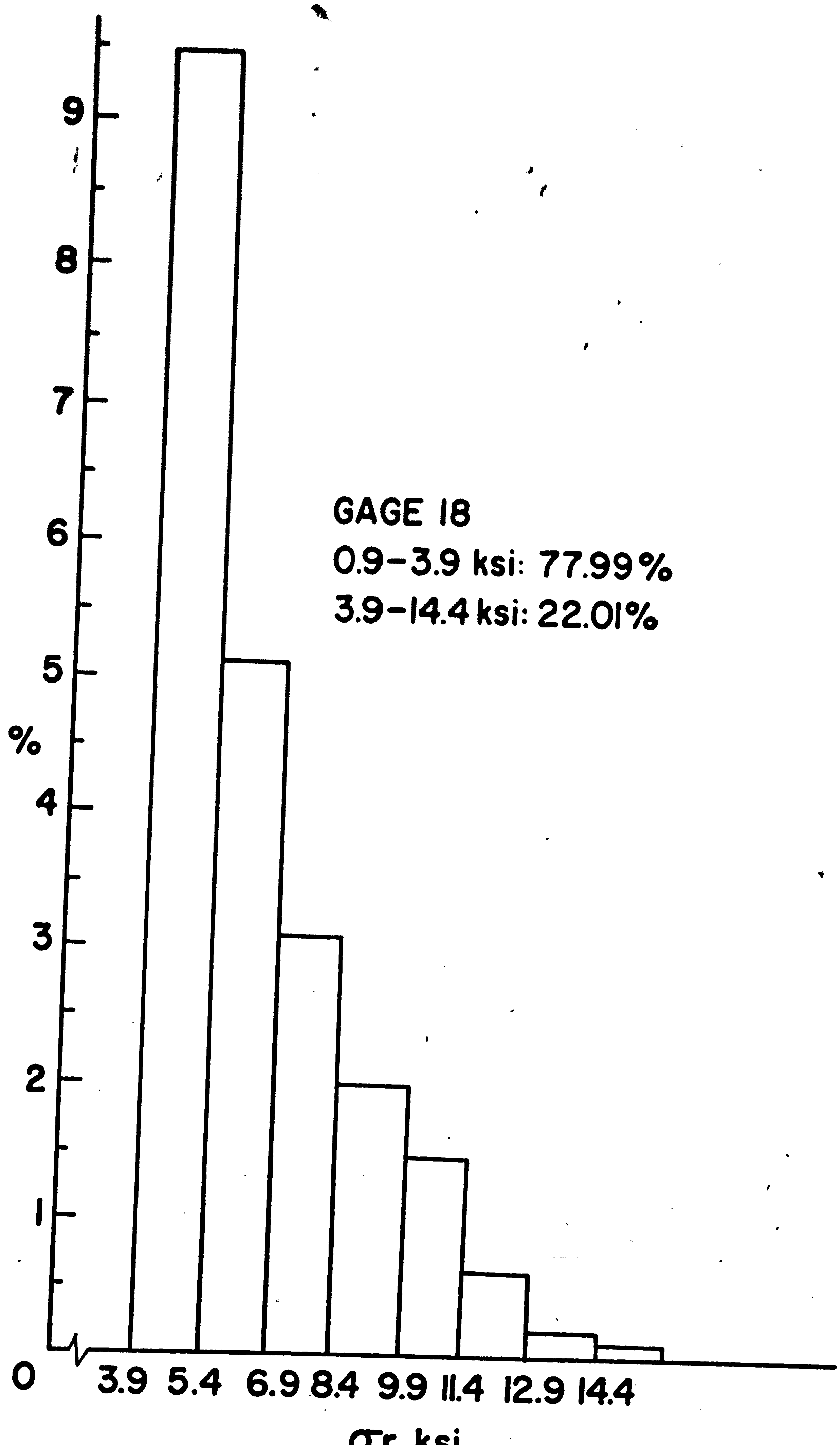


Fig. 31 Histogram for Gage 18 (Tie-Plate RT-5)

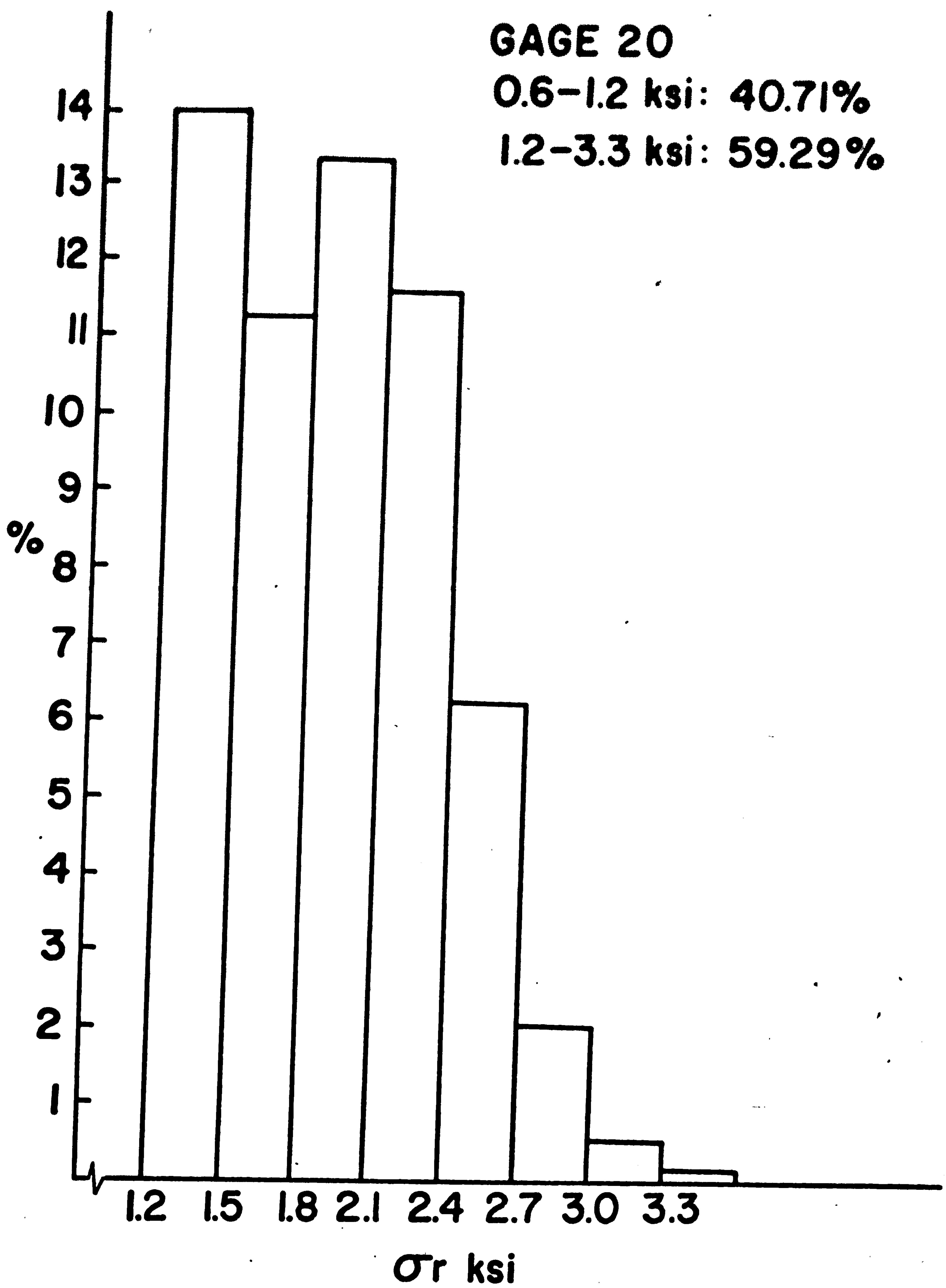


Fig. 32 Histogram for Gage 20 (Bottom Flange, Girder RT)

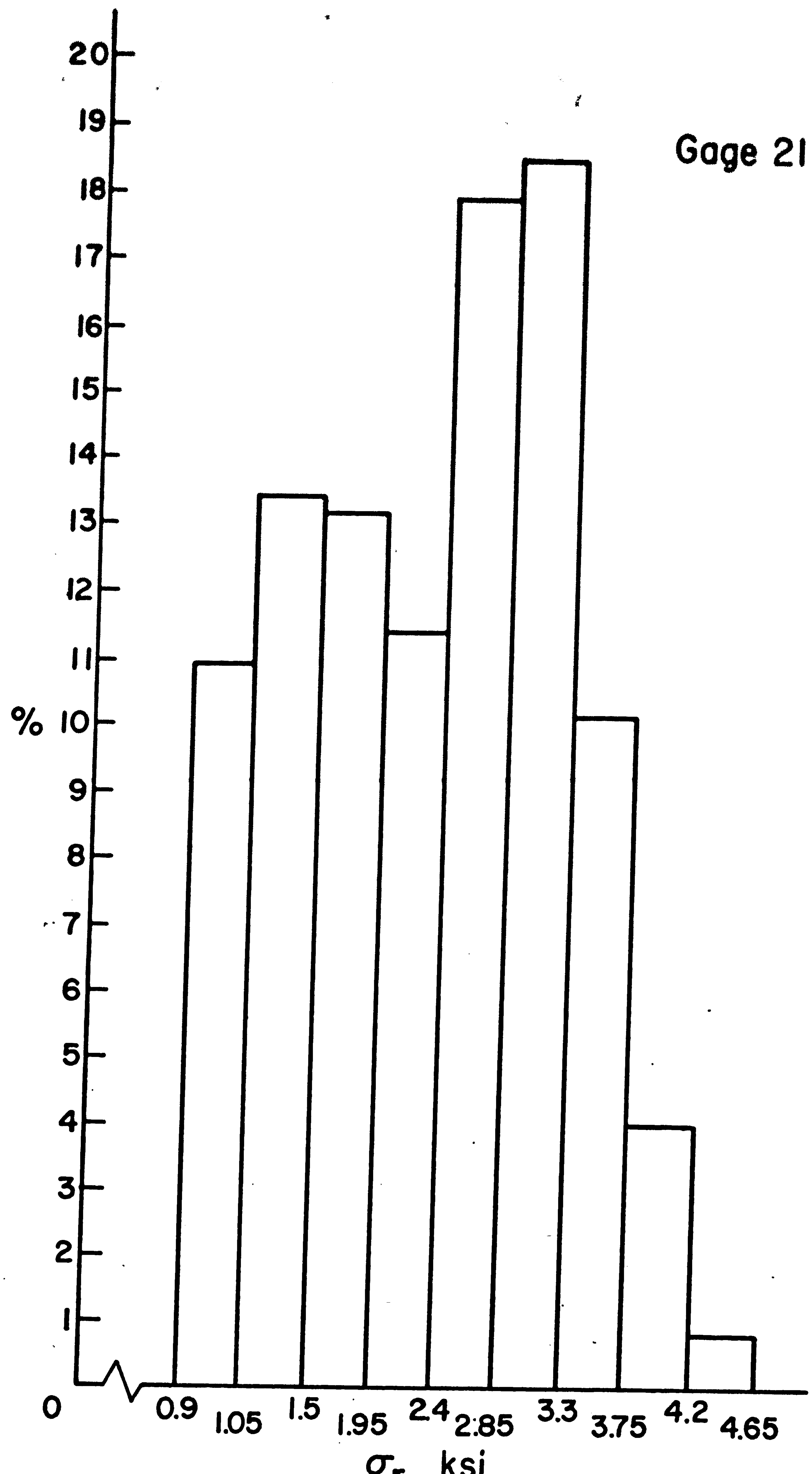


Fig. 33 Histogram for Gage 21 (Bottom Flange, Girder LT)

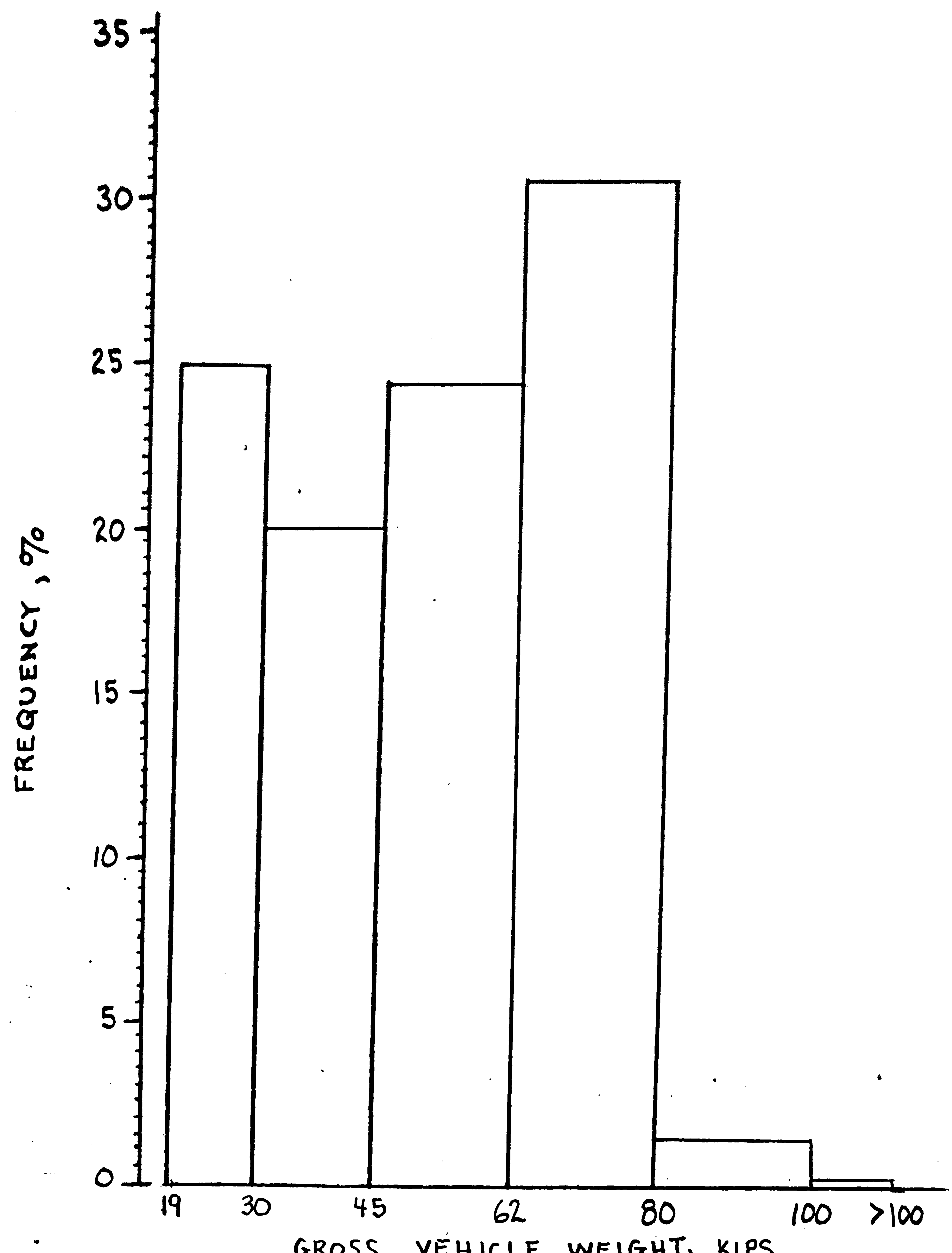


Fig. 34 Histogram of Traffic Flow in Eastbound Lanes of Allegheny River Bridge During Test Period

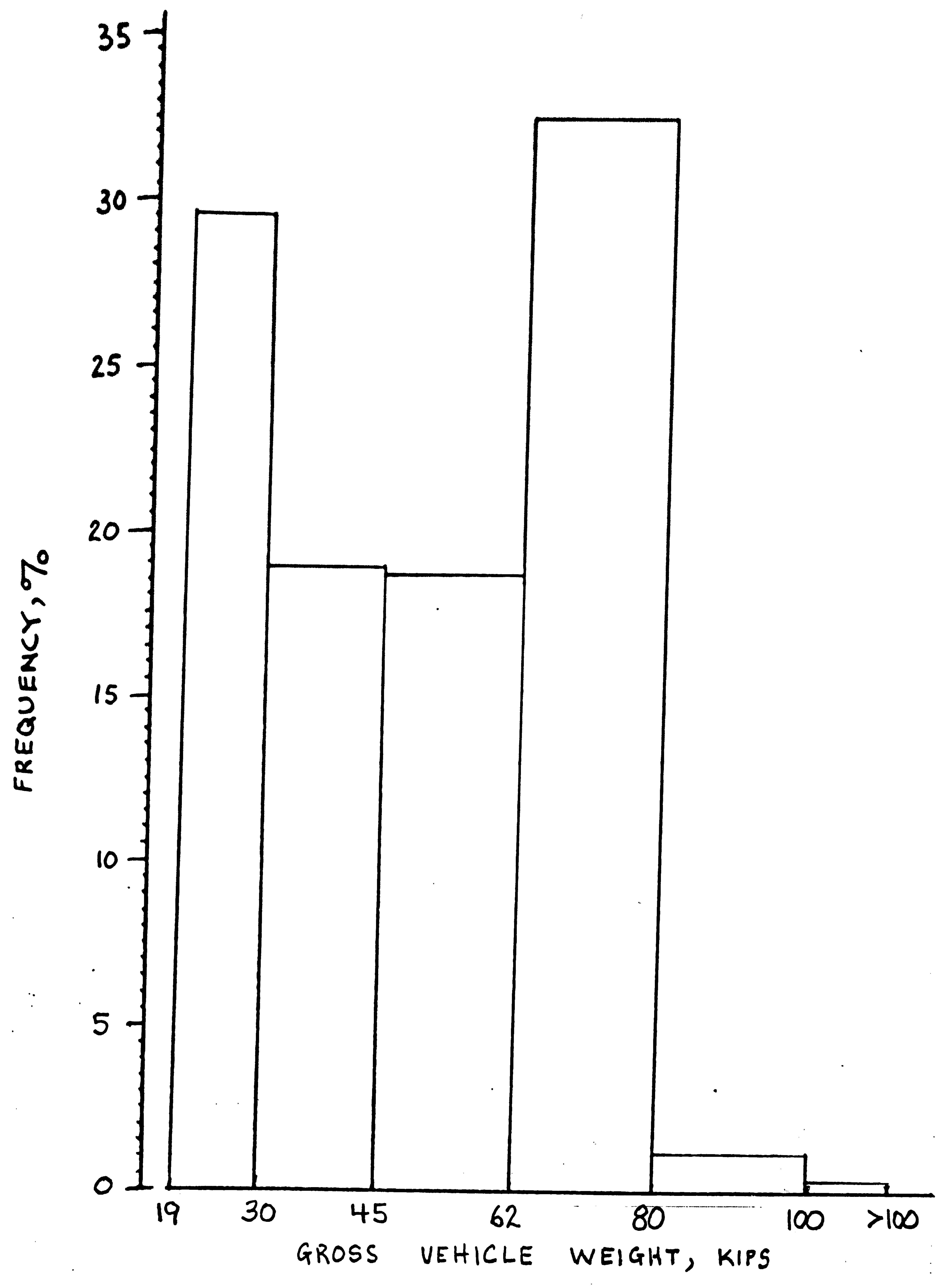


Fig. 35 Histogram of Traffic Flow in Westbound Lanes of Allegheny River Bridge During Test Period

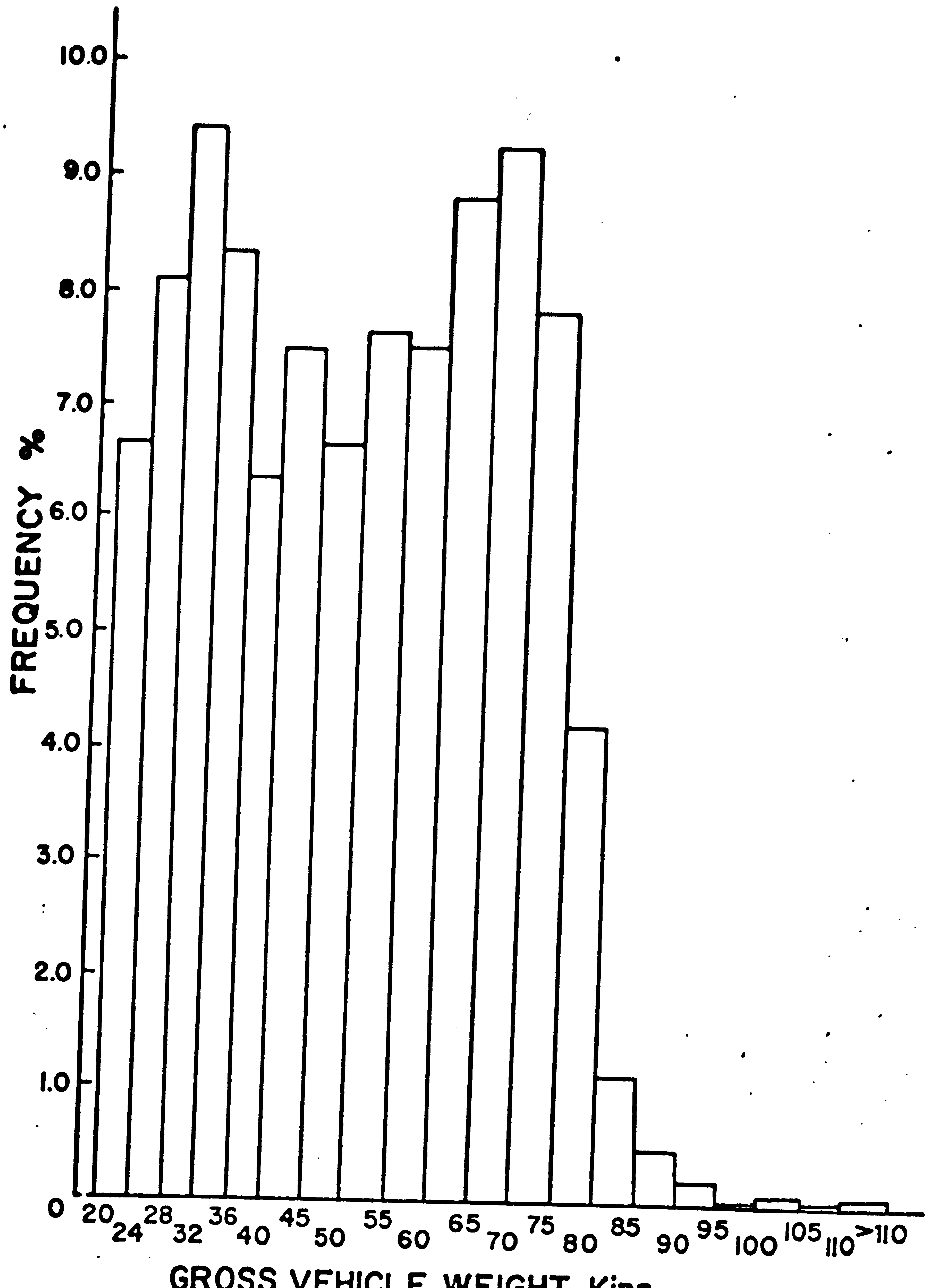


Fig. 36 Results of Loadometer Survey, 1972 (PennDOT, 20 Stations)

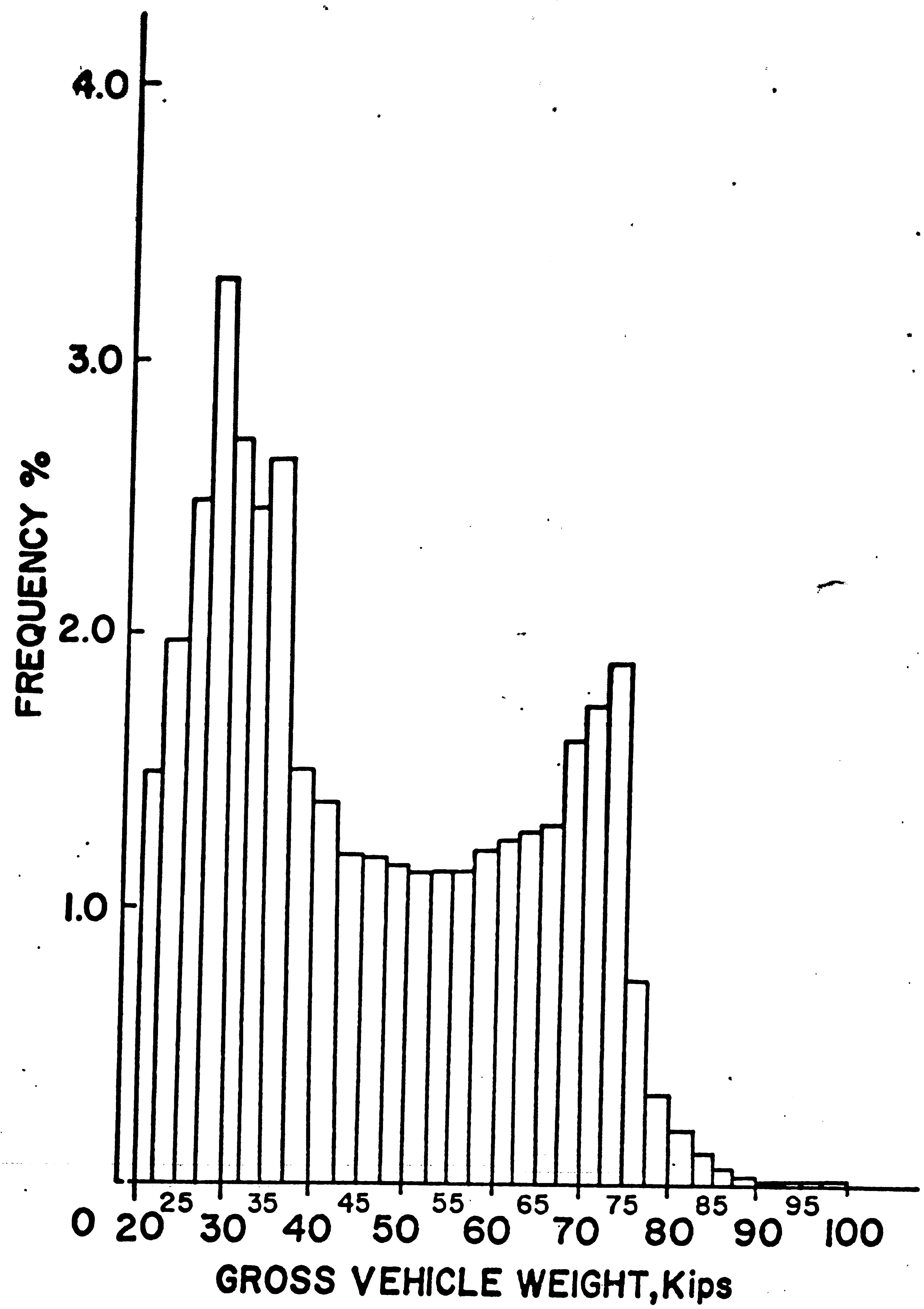


Fig. 37 Results of Loadometer Survey, 1970 (FHWA, nationwide)

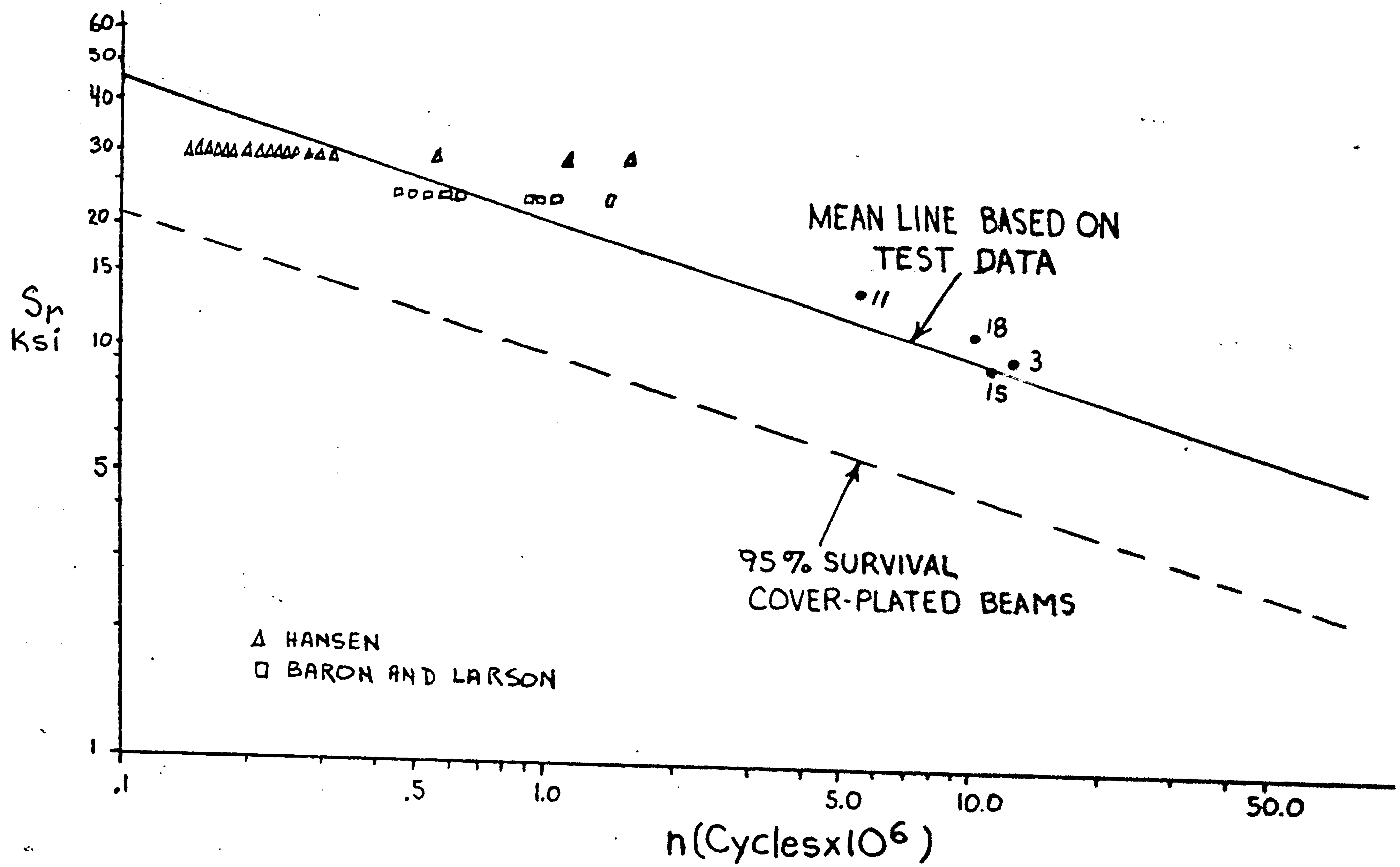


Fig. 38 Correlation of Measured Tie-Plate Response with Laboratory Fatigue Test Results

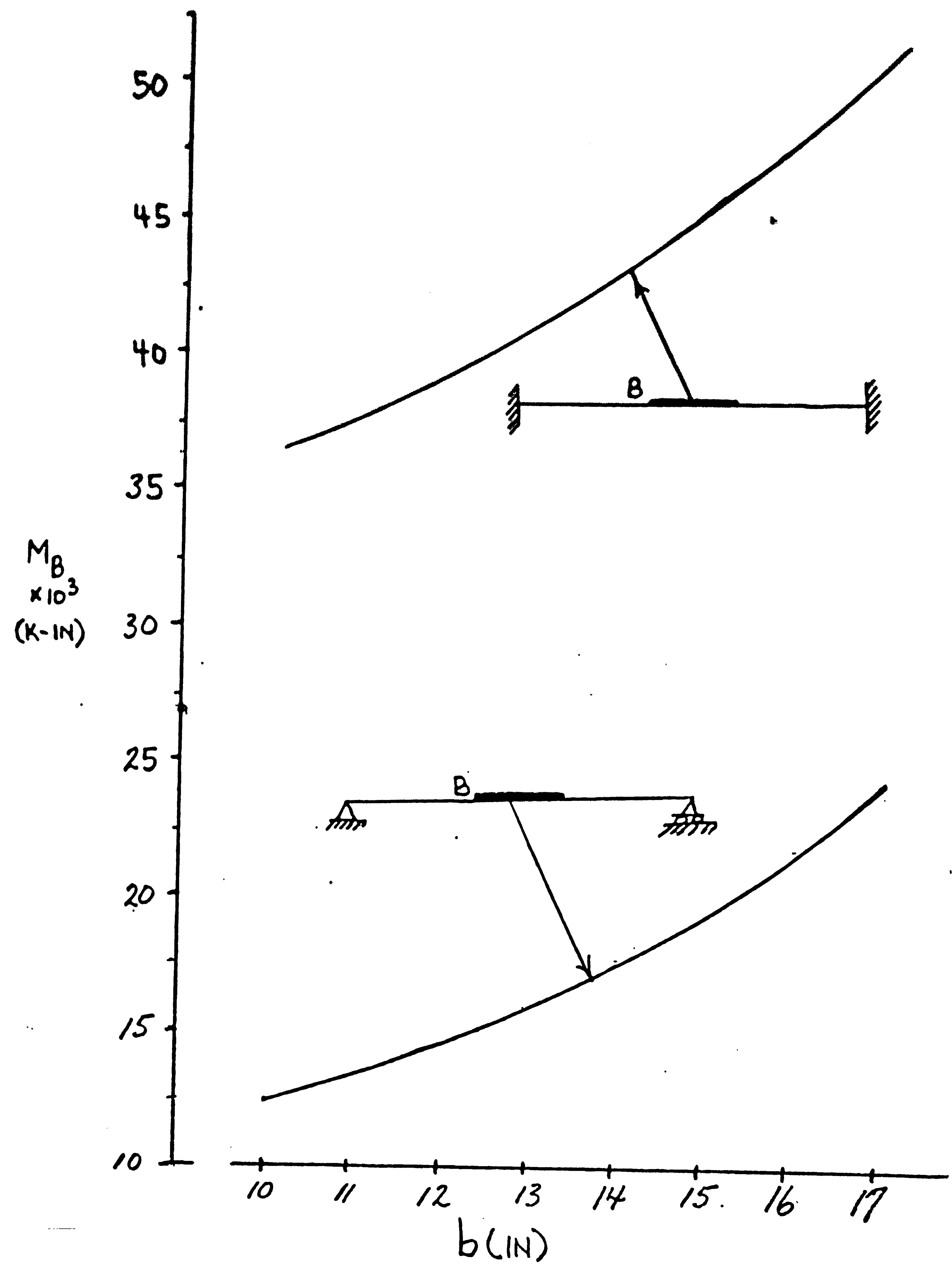


Fig. 39 Effect of Varying Width of Reinforcement Plate on Moment at Point B (Δ ℄ Girder = 1 in.)

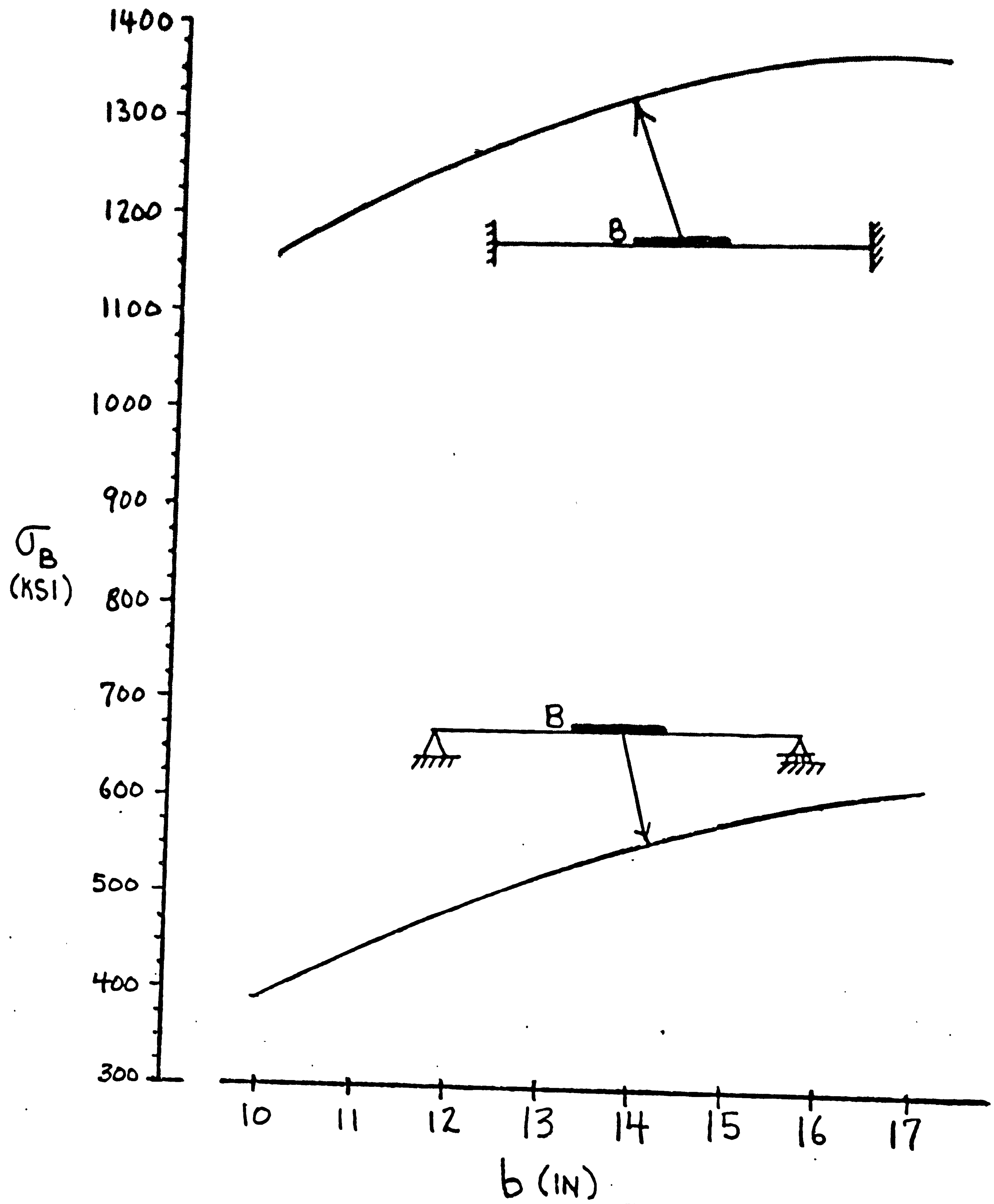


Fig. 40 Effect of Varying Width of Reinforcement Plate on Stress at Point B (Δ ℄ Girder = 1 in.)

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APPENDIX

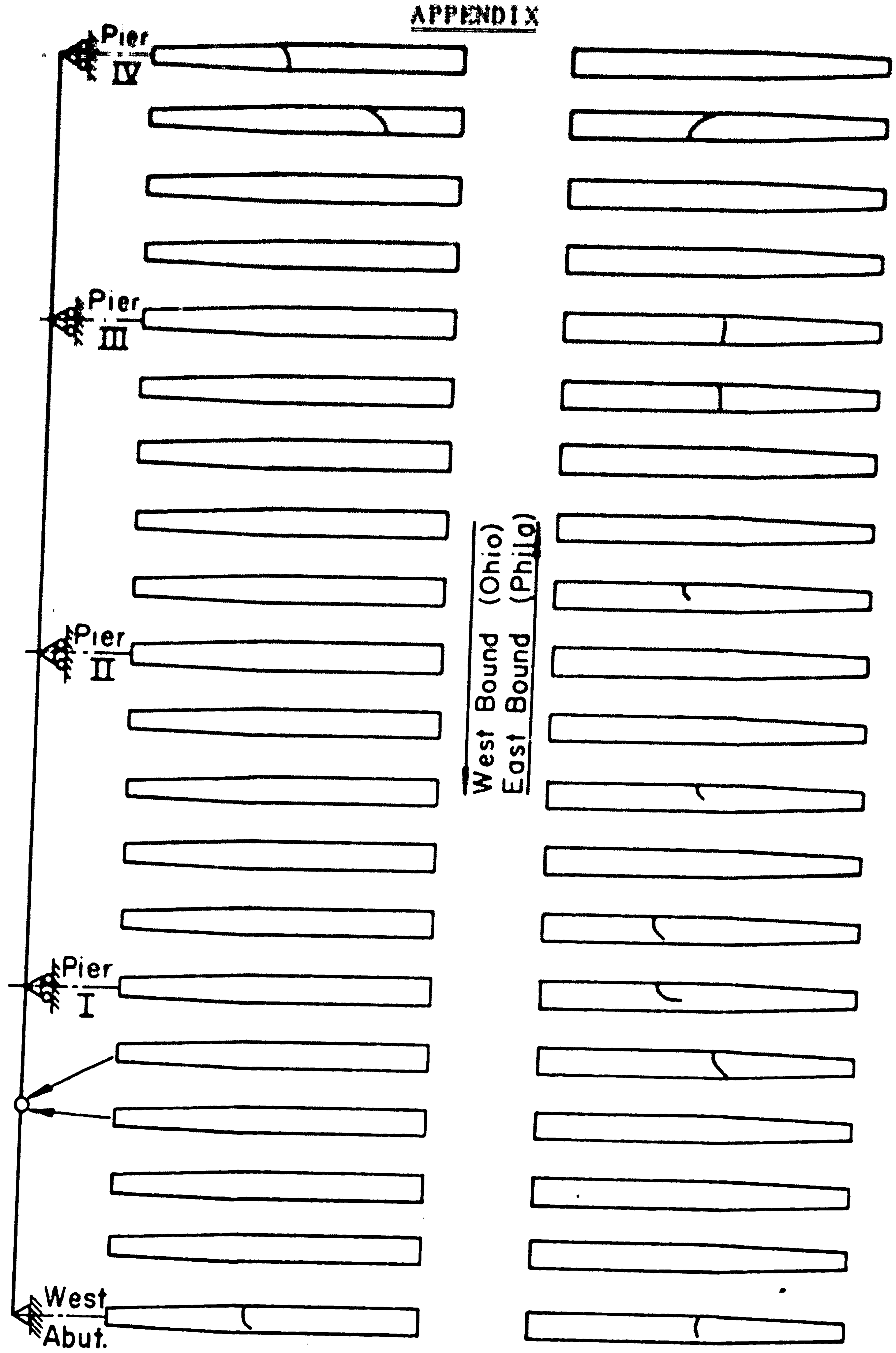

Fig. A1 Cracks in Tie Plates, Allegheny River Bridge

VITA

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The author is an Associate Member of the American Society of Civil Engineers, and has co-authored Fritz Engineering Laboratory Report No. 386.1, "Stress Measurements and Behavior of the Lehigh Canal Bridge" with B. T. Yen and J. W. Fisher, December, 1973.